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SEA WALLS DESIGN CONSIDERATIONS BASED ON A STUDY OF COASTAL PROCESSES AND WAVE ACTION

JOHN B. STETSON

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SEA WALLS

DESIGN CONSIDERATIONS BASED ON A STUDY OF COASTAL PROCESSES AND WAVE ACTION

by

John B. Stetson

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Engineering from Princeton University, 1960.

PREFACE

Sea walls comprise a unique class of structures. The environment in which they are constructed and the unusual forces which they must resist present many special engineering problems. These problems differentiate sea walls from other ground structures such as retaining walls, although some of the basic functions are identical. This thesis is an investigation of the design requirements for various types of sea walls with emphasis being placed on these unusual problems.

Of all engineered structures, the sea wall best illustrates man's struggle to control the forces of nature. To design a structure which will control nature, the engineer must develop an understanding of the natural processes which are occurring along the world's coastlines. This "sea sense" can be partially developed through the study of coastal evolution which has been outlined in Chapter I of this work. It is hoped that the information presented will assist the engineer in complementing natural processes rather than arbitrarily opposing them in achieving a desired result. This is the first step in obtaining a useful coastal protective structure at a minimum cost.

A sea wall design must not only work with the shoreline regimen but also must be capable of resisting the action of waves. The theory of ocean waves has now been developed to the extent that wave characteristics can be predicted with reasonable accuracy for most coastal areas. In the last few years, many model tests have been performed which have correlated the wave characteristics with wave action on various structures. These data with the previously developed theories of wave forces which are essential items of knowledge for the coastal engineer are summarized in Chapter II.

Any coastal structure placed below the high water line will alter the normal littoral processes to some extent. The ability to predict the changes which a structure will cause is extremely important not only in regard to safety of the structure itself but also concerning the adjacent shorelines which may experience adverse effects from the new construction. This complex subject has been too frequently disregarded and is therefore made an integral part of this thesis.

Varying site requirements dictate the use of different types of sea walls. These types are described and factors in selecting a particular type are discussed. Utilizing the basic data developed on coastal processes and wave action, general design recommendations are formulated for each type of sea wall. These recommendations are primarily concerned with the unusual problems inherent in sea wall design. In addition to these specific considerations, the engineer must apply the same sound engineering practice to the foundation design and structural analysis of the sea wall as he would to any other ground structure. Since the combination of a stable beach and a sea wall is the most effective means of coastal protection where natural protective features have been destroyed, a brief section has been included on groins.

It has been assumed that the reader has a basic knowledge of soil mechanics, foundation design and structural analysis. These subjects are well covered in current engineering literature and no attempt has been made in this work to provide basic information in these fields. This must not be construed as an indication that normal engineering principles do not apply to sea walls; on the contrary, they must be carefully applied considering not only the usual forces but also the forces exerted by the sea

and the indeterminant coastal processes.

It is realized that the subject of sea wall design requires a general summary of the entire field of coastal engineering. This thesis is an attempt to compile the most important data on this broad subject in a single volume and to provide a selected list of references for more detailed study of specific areas as required by actual problems in this field.

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CHAPTER I

Coastlines and Shore Processes

A. Introduction

The world's seacoasts are subject to constant change. Coastlines are altered by changes in sea level and by the action of many natural agencies. Sea level changes are generally classified as follows:

(a) Eustatic change - A world-wide movement of the ocean relative to the shorelines. An example of this type is the slow rise in ocean level, estimated at 2 1/2 feet per 100 years, caused by melting of the world's glaciers and icecaps.

(b) Tectonic change - A local movement of the land with reference to the ocean level. Glacial depression of a land mass and volcanic action are two examples of tectonic sea level changes.

While these sea level changes are of importance to the coastal engineer, they are beyond his control and it is the action of other natural agents which is of greatest concern to the engineering profession. These natural agents include waves, tides, currents, run-off, wind, frost action and many others. An understanding of the action of these forces in shoreline evolution is essential if coastal erosion is to be prevented through the construction of protective works. Only through this knowledge can the proper balance between natural forces and coastal protective structures be achieved. Disregard of these shoreline processes will result in structures which disturb the natural equilibrium causing accelerated erosion, unwanted deposition of littoral materials, property damage and frequently destruction of the protective structure itself.

B. Natural Agents

Brief discussions of the various natural agents which are instrumental in coastal evolution are presented in this section. The subsequent sections of this chapter will describe the normal evolutionary patterns of various coastal types under the action of these natural agents.

(1) Waves. Waves breaking on a beach cause great turbulance. The particles thrown into suspension are sorted and transported. Depending on the wave characteristics, particle size and shape, and beach slope either accretion or erosion may be caused. This complex subject is more fully discussed in Chapter III.

Erosion of sea cliffs is caused by the tremendous pressures which can be created by breaking waves, and by impact as loose pebbles and stones are hurled against the face. Pressures up to 8 tons per square foot have been recorded for waves breaking against vertical walls. (Ref. 18, p.30) These pressures act directly on the rock and on air which is trapped in fissures and solution channels. When the wave recedes, the unbalanced pressure within the cliff tends to burst any confining envelope causing rapid erosion. The effect of this wave pressure alone on compact, non-jointed rock is probably not great; however, when combined with the erosion produced by pebble bombardment and by other agents, it can be considered one of the major forces in coastal evolution.

(2) Currents. Currents are caused by oblique waves, mass transport toward the shoreline, tides and winds. Their function in coastal processes is the transport of materials. Loose particles may be eroded by currents, and particles in suspension may be prevented from settling. Transport is effected in four ways: suspension, saltation, rolling and dragging.

(3) Wind. Wind is primarily an indirect force in shore erosion; its energy being transmitted to the coast in the form of waves and spray. In addition, wind acts directly on beach materials to cause dune formations.

(4) Tides. As an agent in evolution of the world's coastlines, the tides may appear to play a secondary role compared to waves, currents and wind. Actually this is not the case since tides are constantly at work while the other agents act spasmodically. Tidal currents do a slow but steady job of scouring, transporting and depositing. This scour is really more of a negative erosion preventing the deposit of alluvium in the areas swept by the tidal currents. The tidal range broadens the front which is susceptible to wave attack. This broadening is both vertical and horizontal. Tidal range also determines the height of wave built bars and spits and the width of the strand zone. Changes in tide level may result in marked differences in the refraction pattern at high and low water thus changing the angle of wave attack and encouraging wave-caused erosion.

(5) Chemical and Biological Processes. This group of natural agents includes solution of limestones, formation of calcareous sandstones, reef construction by corals and the destructive action of various sea organisms.

C. Initial Classification of Coastlines

Initial coastal forms are those caused by tectonic sea level changes. To facilitate the study of coastal evolution, the following classification of initial forms has been developed:

(1) Ria Coasts. These are river systems which have been drowned by the sea. Chesapeake Bay is an excellent example.

(2) Fjord Coasts. Like ria coasts, a fjord is fundamentally due to submergence. A fjord is a glacial valley flooded by the sea. These are characterized by closed depressions which possess a typical U-shaped cross section and are cut off by thresholds of solid rock. The Scandinavian Peninsula presents many illustrations of this coastal form. It should be noted that not all submerged valleys in glaciated regions are fjords. New England is a case in point.

(3) Glacial Lowland Coasts. These coasts are characterized by many glacial accumulation forms such as terminal morains, drumlins, outwash plains, sub-glacial valleys, and kettle holes. These coasts have a distinct appearance and show rapid evolution. Southern New England is a good example of a glaciated lowland coast.

(4) Unglaciated Lowlands. A regular shoreline is the distinguishing feature of this classification. Estuaries, marshy areas and deltas are prevalent.

(5) Coasts Dominated by Structure. There are various subdivisions of this classification but in general structural influences are so important that they provide the only means of classifying certain coast lines. Evidence of recent tectonic movements such as faults and volcanoes or older movements which guide the work of the sea are characteristic of these coast lines. The Gulf of California provides a remarkable example of a structure dominated coast.

D. Sequential Forms

Sequential forms evolve from the initial forms through action of natural agencies. Basically the following coastal forms can be observed:

(1) cliffs and rock platforms

(2) beaches and dunes

(3) marshes and deltas

(4) coral formations

In the first of these forms, erosion predominates; in the latter three groups accumulation is the dominant process.

It will be instructive to consider these forms in more detail to appreciate fully the natural processes which are constantly altering our shorelines.

(1) Sea Cliffs. As waves break directly on a shore line, the land is eroded producing a cliff and a wave cut terrace. As the cliff retreats, more and more wave energy is lost crossing the terrace and is therefore unavailable for erosion. The retreat of the cliff becomes slower and slower until a state of equilibrium would be achieved if sea level were constant. This classic theory of cliff formation is shown in Fig. 2. As previously noted, the sea level is continuously rising and therefore no final cliff line can be predicted. Differential erosion along lines of weakness such as a fault may produce a sea cave or sea arch. When the roof of one of these forms finally collapses, isolated rock columns called stacks remain in front of the cliff. Plunging cliffs are those having no abrasion terrace. The profile and rate of cliff recession depends on the shoreline material and on the agencies acting as discussed in the following paragraphs.

Incoherent sandy materials such as eroded dunes, loess, glacial deposits, recent alluvium and volcanic ash favor rapid evolution. These

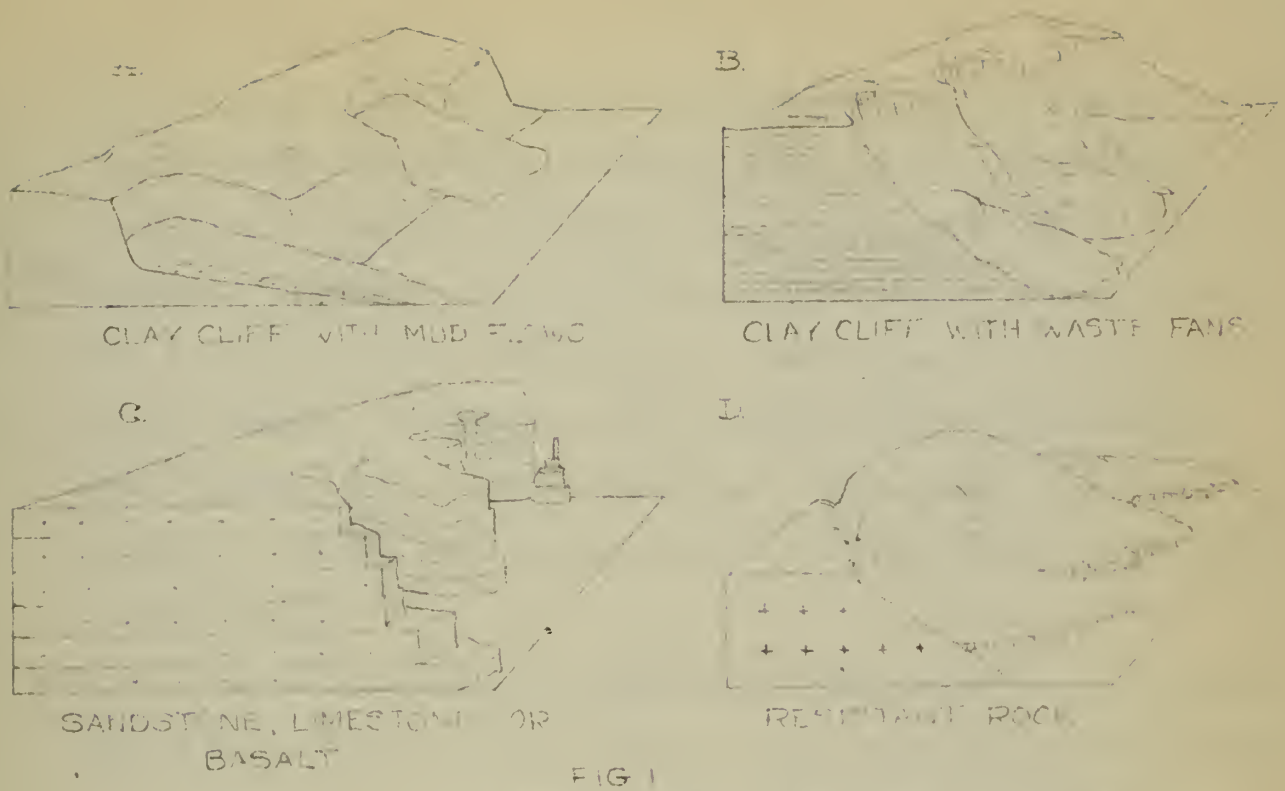


FIG 1
CLIFF TYPES (after Gulcher 1958)

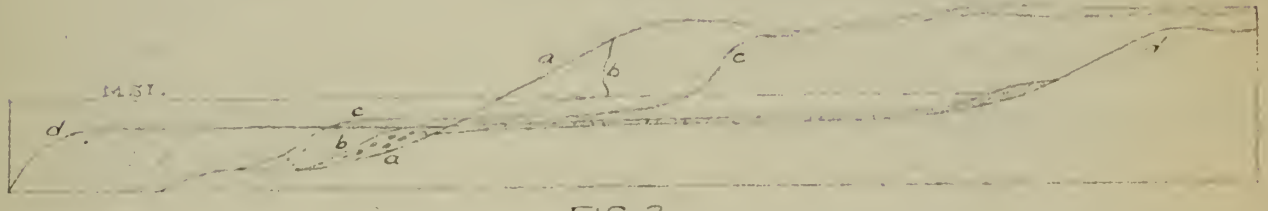


FIG 2
STAGES IN SHORELINE DEVELOPMENT
(after Johnson 1919)

s - SPIT
t - TOMBOLO
bmb - BAYMOUTH BAR
os - BAYSIDE BEACH
bhb - BAYHEAD BAR
cs - COMPLEX SPIT
mb - MIDBAY BAR
bh - BAYHEAD BEACH
hb - HEADLAND BEACH
bd - BAY DELTA

FIG 3
DEPOSITIONAL FORMS
(after Johnson 1919)

deposits never attain equilibrium. Slips result from the tendency of the slope to become less steep and generally follow periods of heavy rain. The sea clears away the debris from the base of the cliff and the cycle is repeated. It should be noted that this debris removal by the sea is necessary for continuation of cliff erosion. Steep slopes may be observed in compact materials of this type where no recent slides have taken place. Some of these cliffs are being eroded at an alarming rate. The cliffs at Holderness, England, for example, are moving back at a rate of approximately 5 feet per year.

Sea cliffs which are predominately of clay materials are characterized by many run-off ravines. Circular slides or mud flows occur resulting in the formation of a convex mass which spreads over the beach at the toe of the cliff. This mass after drying, cracks and is finally removed by the sea. These clay cliffs recede much more slowly than the preceding type, but in both cases continuation of the process depends on the washing action of the sea at the cliff base.

In some areas hard massive rocks occur over impermeable, incompetent rock layers. When the contact surface is tilted toward the sea and becomes saturated with water, conditions are very favorable for a slip which carries great blocks of the upper rock toward the sea. Very low tides also produce a condition favorable to failures in this cliff type since the lower rock is not supported by the sea.

Resistant rocks result in various cliff profiles and modes of evolution. Retreat is much slower than in the above materials. Calcareous rocks, columnar basalts, sandstones and some older shales tend to produce vertical cliffs if the beds are horizontal, vertical or dip

toward the land. When the dip is toward the sea, the profile may follow the dip or take on a saw-tooth appearance. Cliffs in these materials evolve as a result of infrequent and violent slides usually under the action of storm waves and heavy precipitation. Crystalline and metamorphic rocks and hard sandstone evolve to form a convex outline which is usually covered with vegetation over the top two thirds with only the lowest third subject to sea action. These coastal forms suggest stability as the slope is eaten away only at its base without modification above.

Regions of boulder clay and drumlins present complete disorganization of coastal relief as the cliffs retreat. Here all stages of rapid evolution can be observed.

Some of the above cliff forms are shown in Fig. 1.

(2) Beaches and Coastal Dunes. Unlike most sea cliffs, beaches are built, altered and eroded with great rapidity by the forces of nature. Certain qualitative facts concerning these complex processes have been deduced; however, quantitative knowledge of the rates of material transportation, deposition and loss which is essential to the practical application of littoral principles has defied mathematical analysis to date (1960). Because of its great importance to the coastal engineer, the subject of littoral processes is fully discussed in Chapter III.

Through the action of winds, waves, tides and currents, materials are accumulated to form various major shore line features as illustrated in Fig. 3. The most common of these is the bay head beach. This formation usually has an arc-like shape unless there are rocky outcrops in the bay in which case the beach forms a series of geometric arcs between the

rocks. These bay head beaches may be either eroding with replenishment from adjacent cliffs or accreting. In the latter case, several successive beach ridges may be evident. A spit of sand or pebbles occasionally forms perpendicular to the axis of a bay half way between the head and the mouth. This is caused by retardation of the waves moving into the bay and is called a mid bay bar. The bay mouth bar is a much more common formation and results from a longshore drift. The spit grows in the direction of the drift and the mouth, whose width will depend on the volume of water passing through it, will become aligned parallel to the coast.

Another form of spit is the offshore bar which is often originally formed on a reef or rock ridge. These bars are normally driven shorewards and may be transformed into barrier beaches. Where important streams are blocked by these formations, one or more channels must remain open. The world's finest offshore bars are those forming Cape Hatteras.

The tombolo is another classic accumulation form. This is defined as a spit which connects an island to the neighboring coast. These take many forms all of which can be explained by the refraction of waves by the island mass.

The growth of a spit or bar is usually accomplished by the addition of hooks or lateral ridges. Widening is always connected with lengthening. During periods of strong winds and swell with an abundant supply of material, new storm beaches will be built up causing extension of the spit toward the open sea and parallel to the shoreline. The root of the spit may be eroded to provide this material. The transient dividing line between erosion and deposition is called the fulcrum.

Dunes are natural formations of sand behind a beach which provide excellent protection from storm attack. Because of their transient nature, however, dunes have become a major problem in some areas. Basically, dune formation depends on (a) an adequate supply of beach sand, (b) an absence of obstacles to seaward permitting strong winds and (c) absence of vegetation on the beach. Permanent dunes can exist only beyond the reach of the sea and depend on the growth of vegetation to stabilize the deposited sand. Although dunes give the impression of great depth, they may actually only constitute a relatively thin covering on the pre-existing relief. Frontal dunes which are formed just behind the beach remain weak, are frequently breached by wind "blowouts" and may disappear altogether. In other cases, these frontal dunes remain in place and continue to feed other dunes to shoreward. Two plants which are instrumental in dune construction are the following:

(a) Agropyrum junceum, a salt tolerent beach grass.

(b) Psamma arenaria which, while not able to live in a salt atmosphere, is interesting in that it requires a constant supply of fresh sand for growth.

Carex arenaria, crucianella maritima, and various lichens are effective plants in fixing dunes which have been previously formed.

(3) Marshes and Deltas. Tidal marshes are low areas approximately at sea level. This type of sequential form is common on all low coasts and is originated by silting at the sides of a sheltered estuary, behind a protective spit or bar or at the head of a bay where no major river flows. The deposited mud acquires great stability when colonized by marsh vegeta-

tion. As the vegetation becomes complete and the mud dries, the character of the marsh changes. Upward growth of the marsh from the deposition of mud at high water and from peat formation leaves only tidal creeks which are kept clear of vegetation by constant tidal action. As the marsh continues to mature, the system of channels disintegrates and a flat surface dominates the scene. This is the typical "salt marsh." Its evolution is complicated by the compressibility of the peat and by occasional shoreward migration of coastal forms such as sand bars.

Deltas are river mouth formations where alluvium accumulates rather than being completely redistributed by the sea. The Mississippi and Nile Deltas are excellent examples of these land extensions. The sea or lake into which the river flows causes erosion and redistribution of the material. Abandonment of a part of the delta after the river cuts a new channel is the most frequent cause of erosion. Redistribution consists essentially of building up spits or bars.

(4) Coral Structures. Sequential forms of this classification are distinct from all others since the coral formations are of organic origin and are not principally dependent on the action of ordinary coastal processes. Corals build reefs only in limited areas where temperature, depth, water movement, salinity and turbidity are favorable. Light is essential and therefore reef formation occurs principally between low tide and 15 fathoms. The most favorable temperature range is 25 to 30° C. Coral reefs may present dangers to navigation but more frequently these natural formations act as breakwaters providing a high degree of coastal protection. They have also been proven a valuable source of aggregate for

shore construction in areas where coral formations are prevalent.

E. Shoreline Evolution

Having discussed the various basic initial and sequential coastline forms, several general statements regarding shoreline evolution can be developed. In general, the sea tends to regularize shorelines. All initial types of coast except the unglaciated low lands are irregular. Refraction of ocean waves concentrates wave energy on the headlands which are therefore more rapidly eroded than the lands bordering on adjacent bays. Spits are formed by longshore currents and the coastline is smoothed out. When this point is reached, the coast tends to retreat parallel to itself forming the abrasion platform which promotes ultimate stability of the shoreline. This general theory has been verified by studies of morainic coasts such as that of New England where evolution is fairly rapid.

The sea is also capable of differential erosion especially in the initial stages of coastal evolution where weakened forms are attacked leading to an irregular coastline. This is true not only for minor details such as sea caves but also for the general erosion pattern of the coast. It can be said therefore that initial structures exert a great control over coastal development.

Finally, it should be noted that coastal evolution may be interrupted by many factors resulting in complete changes in the predicted development. Excellent examples are the closing of a strait by a spit formation, intermittent uplift of the land mass and frequently the introduction of an artificial protective structure which upsets the equilibrium of nature.

CHAPTER II

Waves and Wave Forces

A. General

In the long history of engineering, no single force has so captured man's imagination as that embodied in ocean waves. Some of the world's greatest engineering triumphs and defeats have resulted from man's attempts to control these forces. The study of wave theory, generation, forecasting and force constitutes a complete field of engineering in itself. No attempt will be made to condense the wealth of specific information available in the vast literature on the subject since in condensation much of the essential data would be lost. However, an outline of the basic areas of knowledge concerning waves and wave forces will be provided. This outline together with the listed references on the subject will serve as a guide to solution of engineering problems concerning wave action. This field of science is not new; however, great progress has been made since World War II. New methods and revisions of old methods are constantly being incorporated into the knowledge of wave action. It is essential therefore that engineers concerned with shore protection structures keep up with these advances to insure that the latest engineering tools are utilized in the design of water front structures.

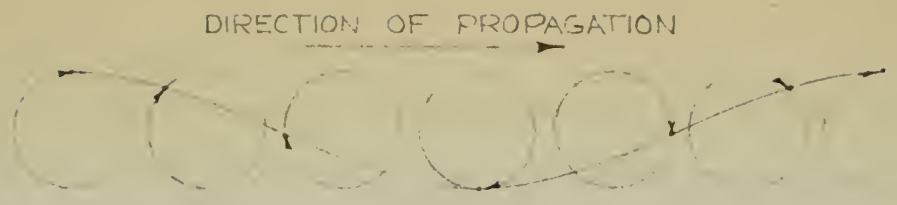
B. Wave Classification and Theory

Waves are classified on the basis of wave period. This characteristic is defined as the time interval between the passage of successive wave crests at a fixed point. The major divisions are as follows:

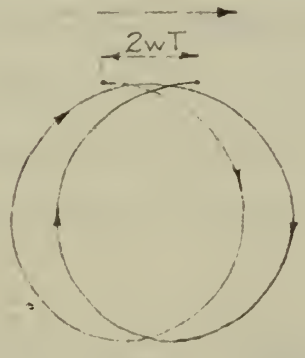
Capillary waves	less than 0.1 sec.
Ultra gravity waves	from 0.1 to 1.0 sec.
Ordinary gravity waves	from 1 to 30 sec.
Infra gravity waves	from 30 sec. to 5 min.
Long period waves	from 5 min. to 12 hours
Ordinary tides	12 hours to 24 hours
Trans tidal waves	24 hours and greater

The range of periods covered by the above classification constitutes the complete spectrum of ocean waves. Most of the wave energy is contained in the following two divisions: ordinary gravity waves and ordinary tides. These two period ranges are therefore of greatest importance in engineering applications. It is through gravity waves that the energy of storm winds is transmitted to the world's shorelines. The importance of the tides, aside from nautical considerations, lies in their effect on water level, i.e., the point of application of gravity wave forces, and in their ability to prevent deposition as previously discussed. In some areas of the world a special type of long period wave of great height presents major engineering problems. These are the so-called "tidal waves" or "tsunamis" caused by sudden displacement of the ocean bottom during submarine earthquakes. Japan and the Hawaiian Islands have frequently experienced their great destructive power.

There are two general theories of wave motion which have been developed to explain mathematically the behavior of ocean waves as they approach a shoreline through deep water into and through shoaling water. The first of these is the trochoidal theory based on an oscillatory movement of particles in the wave. Each particle is assumed to describe a



WATER WAVE OF SMALL AMPLITUDE
BASIS FOR NORMAL WAVE FORMULAE



w = MASS TRANSPORT
SPEED = $(\pi \frac{H}{L})^2 C$

IRROTATIONAL THEORY

FIG 4
PARTICLE MOTION IN WAVES

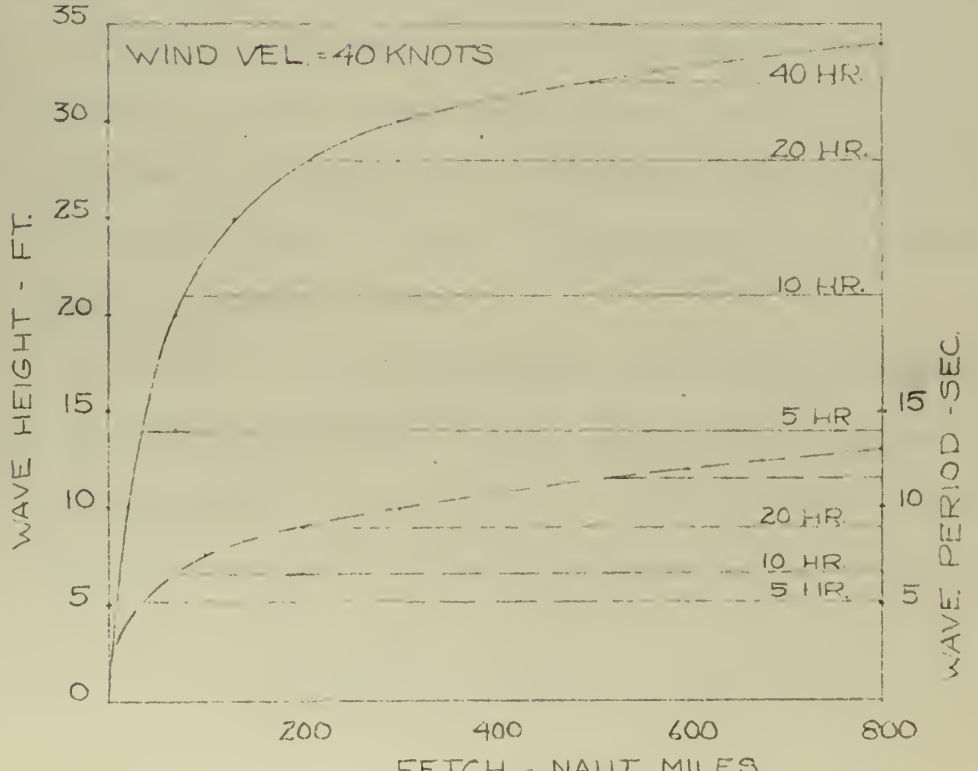


FIG 5
WAVE HEIGHT AND PERIOD AS FUNCTIONS
OF DISTANCE AND TIME

circular orbit about its normal location as shown in Fig. 4A. This theory does not predict mass transport in the direction of wave propagation although observations by the Beach Erosion Board have shown that mass transport does occur. The theory despite this limitation is often used in engineering calculations because of its mathematical simplicity. The extension of this theory, which was originally based upon water of infinite depth, to situations involving finite depths by assuming that the circular patterns become elliptical is not sound. This conclusion has been validated by tests at the University of California.

The second approach called the "irrotational theory" is based on particle motion as shown in Fig. 4B. The particle velocity is assumed greater in its forward motion at the wave crest than in its backward movement in the trough. There is therefore a progressive particle movement in the direction of wave propagation. This movement is known as "mass transport." Laboratory experiments have confirmed the conclusion that the particle orbits are open rather than closed as per the trochoidal theory. The irrotational theory most nearly represents actual wave action; however, it is much more difficult to apply and has therefore not been as widely used as the earlier trochoidal theory.

In general, if ocean waves are of small amplitude compared to their length and to the water depth, the wave profile closely approximates a sine curve. Waves generated in deep water are usually of this type and their properties can therefore be expressed mathematically as follows:

$$L = CT$$

$$C^2 = \frac{gL}{2\pi} \tanh 2\pi \frac{d}{L}$$

where L = wave length, the horizontal distance between two corresponding points on successive crests

C = wave velocity

H = wave height, the vertical distance to a crest from the preceding trough

T = wave period in seconds

d = depth of water

g = acceleration of gravity ($32.2'/sec^2$)

In deep water, where the depth is greater than half the wavelength, the hyperbolic tangent function in the above formula is nearly equal to unity and therefore the velocity, C, can be expressed as

$$C = \sqrt{\frac{gL}{2\pi}}$$

and the wavelength, L, can be approximated by $L=5.12T^2$. Where the water depth is less than 1/25 the wavelength (shallow water) the term $\tanh \frac{2\pi d}{L}$ becomes nearly equal to $2\pi d/L$ and the equation for wave velocity reduces to

$$C^2 = gd$$

Between these two limit situations (transitional water) the formula is used as written.

The energy per unit crest width in deep water, E_T, is given by the following formula:

$$E_T = \frac{1}{8} \rho g H^2 L \left(1 - M \frac{H^2}{L^2}\right)$$

where ρ = mass density of water

M = a function of d/L

$$M = \frac{\pi^2}{2 \tanh^2(2\pi\, d/L)}$$

In nature, an infinitely long series of waves does not exist. Instead groups or trains of waves are generated and travel over the ocean surface. In these groups, the lead wave is gradually reduced in height as its potential energy is utilized in imparting kenetic energy to the undisturbed water ahead of the wave train. The leąd wave will finally disappear while at the same time a new wave is formed at the rear of the group. The group velocity is therefore less than the velocity of individual waves and has been shown to be

$$C_G = \frac{1}{2} C \left[1 + \frac{4\pi d}{L} \Big/ \sinh \frac{4\pi d}{L}\right]$$

In deep water $\sinh \frac{4\pi d}{L}$ becomes very large and C_g is equal to $\frac{1}{2}$ C. In water of depth less than L/25, termed shallow water, $\sinh \frac{4\pi d}{L}$ approaches $\frac{4\pi d}{L}$ and the group velocity therefore approaches the velocity of the individual waves. It is obvious that between these two limits, called transitional water, the group velocity gradually increases as the water depth decreases.

The relationships outlined above are essential to the understanding of wave behavior.

C. Wave Generation and Decay

Ordinary gravity waves are created by the wind. For the transfer of energy from atmosphere to water, the existence of a wave although of very small height, is essential. It can safely be assumed that the gustiness of the wind assures a steady supply of the small waves. The wind exerts a greater pressure on the back of these small waves than on the front. This unbalanced pressure together with a frictional drag over the water surface transfers energy causing the waves to increase in magnitude. An area over which the wind is blowing and generating waves is known as a

fetch. Wave growth within this area is governed by at least three factors: wind velocity, wind duration, and fetch length. When a wind of constant speed blows over the fetch area, the wave height and period increase until a steady state has been reached at each fetch distance. This steady state indicates that no matter how long the wind blows, no change will occur in the waves at that point. This steady state gradually expands over the entire fetch. The relationship between wave growth, fetch length and wind duration is shown in Fig. 5. It can be seen that after 5 hours duration, a steady state has been reached to a distance of 35 miles. If the wind continues long enough, waves leaving the fetch front will have attained the maximum size possible for the fetch length. Greater duration will not produce larger waves and wave size is therefore limited by the fetch length. The fetch length can be limited by a coast line or by the characteristics of a wind system over the open ocean. Normally, however, duration rather than fetch is critical in determining maximum wave growth. These waves being generated within a fetch are called a "sea."

Upon leaving the fetch, the sea is termed "swell." If a calm area is encountered, the swell will be acted upon only by air resistance against the wave front which causes a loss of wave energy with a decrease in height, and a corresponding increase in wave period. This process is known as wave decay and can be predicted quite accurately. When opposing or following winds are encountered, the decay relationships must be modified by utilizing an effective decay distance rather than the actual surface distance.

D. Wave Forecasting

Prior to World War II, little information was available on deep

water wave generation and decay. It became essential during the war to predict wave activity in areas where amphibious operations were to take place. A semi-theoretical approach to the problem was developed by Sverdrup and Munk which combined classical wave theory with available data. This method which permits forecasting wave growth in a slowly moving or stationary wind field having more or less constant wind speed and direction, was published in 1947 (Ref. 23). A subsequent publication by the U.S. Navy Hydrographic Office (Ref. 24) summarized the techniques involved in utilizing this theory and provided the necessary curves. As additional wave data became available, revisions to the method were incorporated (Ref. 8) and later a graphical method was proposed by Wilson (Ref. 27) which permits wave prediction in a wind field of variable wind velocity and variable speed of progression over the ocean surface. When forecasting deepwater waves by the above method, use is made of the term "significant wave." The significant wave height is the average of the heights of the highest one-third waves in a wave train or wave record. The mean period of the significant wave is termed the significant period. This "significant wave theory" or SMB (Sverdrup-Munk-Bretschneider) method includes empirical relationships for the decay of waves in deep water. These relationships are available in the form of practical curves (Ref. 4) A linear relationship exists between the significant wave height and the mean wave height in a train of waves as follows:

$$H_m = 0.624\ H_s - 0.015$$

The mean wave height, H_m, has been related statistically to any other height which may occur in the wave group. This relationship has been presented graphically (Ref. 4, p. 19) and permits determination of a wave height which

will more nearly represent the maximum to be found in any wave group.

The greatest decay of deepwater waves takes place in the first few hundred miles. If, however, the swell enters shallow water, the rate of decay increases sharply as a result of bottom friction and percolation in a permeable sea bottom. An over-all bottom friction factor has been developed which includes both of these effects. (Ref. 9)

It should be realized that wind never generates well-defined wave trains with uniform heights, periods or wave lengths. Instead, a whole spectrum of waves is formed ranging from ripples to mountainous waves in a storm sea. A method has been developed by Pierson, Neumann and James (Ref. 20) to forecast this spectrum. Once the wave spectrum is known, many properties of the ocean waves at any point can be deduced including heights, height distributions, wave periods, and period ranges. This method also extends to the forecasting of waves in the decay area.

Both of the above methods have their relative advantages and disadvantages. It must be borne in mind that in each case the accuracy of the resulting information is a direct function of the data utilized in the forecast. It appears that application of Wilson's graphical method to both of the above forecasting procedures will result in greater accuracy. It is interesting to note that Refs. 5 and 6 provide North Atlantic Coast Wave Statistics developed by the significant wave theory and the wave spectrum method respectively.

Less information is available concerning the generation of waves in shallow water than that presented above on deep water waves. A numerical procedure has been developed, however, for computing wind waves in shallow water of constant depth which can be verified by wave data. This procedure

can be extended to a bottom of constant slope such as the continental shelf by segmenting the bottom into elements each assumed to have a constant depth equal to the mean depth of the segment. (Ref. 9)

A third area of importance in wave forecasting concerns the prediction of hurricane generated waves. There are three fundamental differences between generation of waves under hurricane conditions and under normal wind action. These are as follows; (a) Winds within a hurricane are not constant in speed. (b) Winds within a hurricane are circular in direction and (c) The hurricane moves over waves generated at various angles to the path of the storm. In predicting hurricane formed waves, the most satisfactory method is the Wilson graphical approach.

Since the forecasting of waves is a relatively young science, all of the above methods are constantly being revised as additional calibration data is made available. With these improvements in methods of forecasting, more and more statistical data is becoming available to the coastal engineer particularly concerning waves to be anticipated in the offshore areas of the United States. The brevity of the above discussion on wave forecasting should not be construed to mean that the compilation of statistical data is a relatively easy task. It is exactly the opposite, requiring a high degree of technical knowledge and skill coupled with many months of analysis of synoptic charts and other data sources. It is certainly to the coastal engineer's advantage therefore to know the sources of statistical data for the area in which he is working. The references in this section will provide some leads in this field.

E. Shoal Water Effects

As indicated above, offshore waves at any point can be predicted with a fair degree of accuracy. As the swell enters water of depth less than one half its wave length, bottom effects become evident. The first noticeable change in wave behavior is a bending or refraction tending to cause the wave crests to parallel the bottom contour lines. This refraction is extremely important to the coastal engineer not only because the direction of wave attack will affect the final wave forces applied to a protective structure, but also because this bending results in zones of wave convergence which should be avoided if possible in locating shore structures. In addition, wave refraction influences the direction of littoral drift along the shore. This important topic is more fully discussed in Chapter III.

The prediction of changes in wave direction resulting from bottom topography from the L/2 contour to the breaker line is accomplished through the use of refraction diagrams. In general, the following two methods are in common use for constructing these diagrams: (a) the wave-crest method in which successive positions of the wave crests are first plotted. Orthogonals, lines perpendicular to the wave crests between which transmitted wave energy is constant, are then drawn. (b) the crestless method in which each orthogonal is plotted directly by determining its shoreward deflection as it crosses successive bottom contour lines. The first method is normally used where relatively flat banks and gentle slopes occur in the underwater topography and where the exact alignment of bottom contours is difficult to obtain. The crestless method is utilized where

other conditions of bottom topography occur. The mechanics involved in the construction of refraction diagrams is presented in detail in Beach Erosion Board Technical Report No. 4 and in U.S. Navy Hydrographic Office Publication No. 605.

Another phenomenon involving change in wave direction is diffraction in which waves are induced in a sheltered region formed by a breakwater or similar barrier which interrupts part of an otherwise regular wave train. In this process, unlike refraction, wave energy is transferred laterally along a wave crest. This factor becomes of importance primarily where shore protection structures are needed in an area already partially protected by a breakwater. References 4 and 25 are recommended in this connection.

In addition to changes in direction as a wave travels into shoaling water, the wave length and velocity decrease. With the wave period remaining constant, conservation of wave energy requires that the wave height increase. The wave steepness H/L increases until the wave breaks at approximately H/L = 0.1418 and breaks again several times as it approaches the shore as shown in Fig. 6 until finally breaking in a single plunge or by spilling over itself for an appreciable time. On a steep beach, partial breaking seldom occurs and a single plunge or surge characterizes the final demise of the wave form.

Beyond the breaker line refraction theory no longer holds and refraction diagrams must end at this point. In general, it appears that breaking will occur in water of depths approximately equal to 1.28 times the wave height. Breakers are classified in three types as follows:

(a) Spilling breakers in which the wave crest becomes unstable in

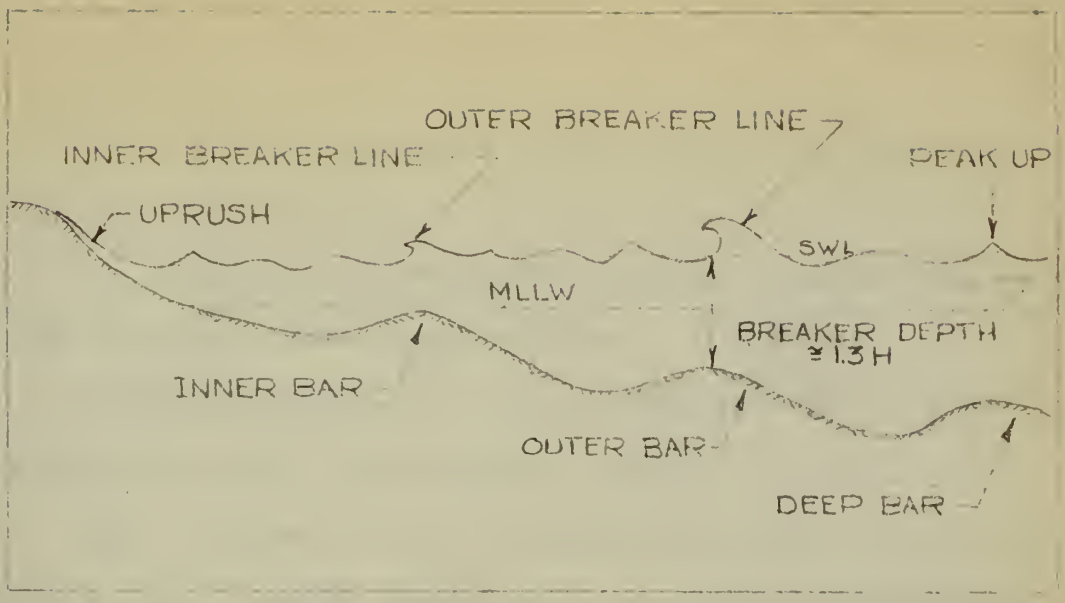
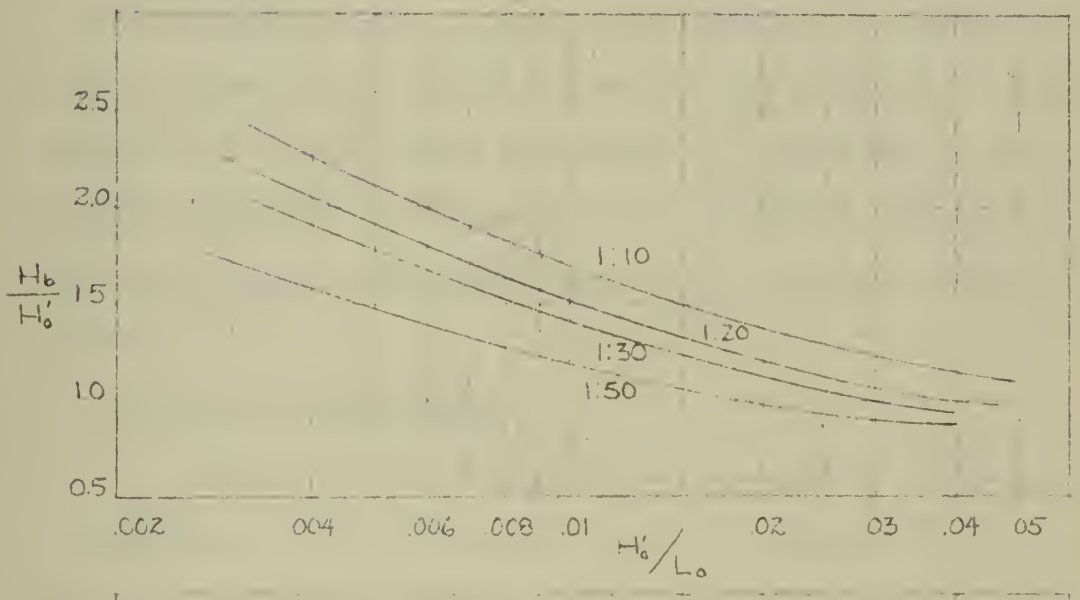


FIG 6
WAVES IN THE BREAKER ZONE (Ref 4)



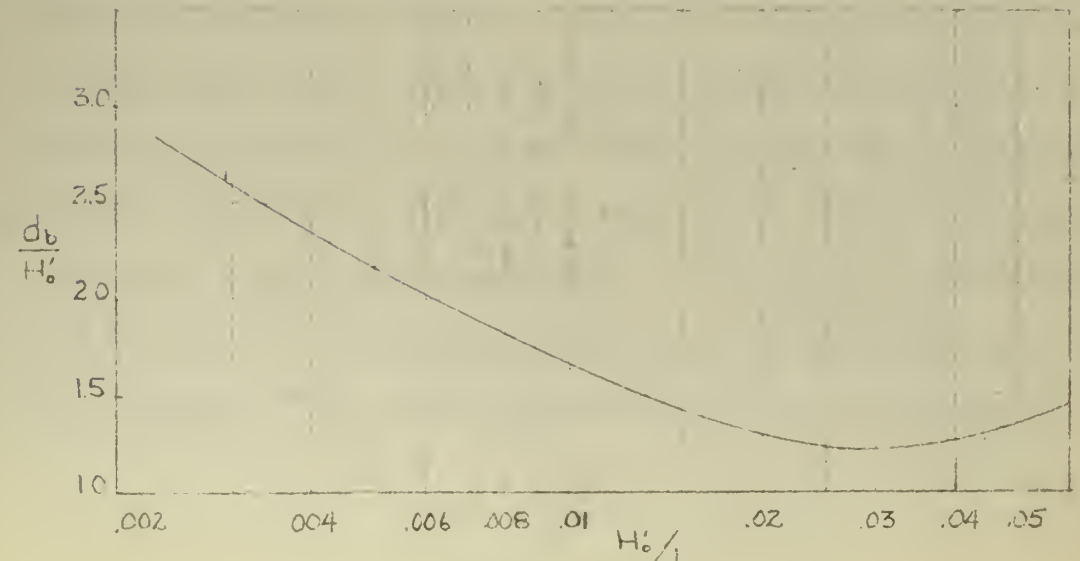


FIG 7
IVERSON CURVES

a mild fashion with the appearance of white water at the crest which expands down the wave face,

(b) Plunging breakers in which the crest overshoots the body of the wave to project ahead of the wave face,

(c) Surging breakers in which the wave face becomes unstable over its entirety in a large scale turbulent fashion.

Tests performed at the University of California (Ref. 16) have provided a fund of useable information on the complex transformation of a wave into a breaker. Of particular value is the resulting correlation between breaking point, deepwater wave steepness, and beach slope. Curves, Fig. 7, showing these relationships have been adopted for practical application by the Beach Erosion Board (Ref. 4). Knowledge of the wave breaking point is essential in the design of any protective structure since the final wave energy dissipation depends on the wave characteristics at this point.

F. Selection of Design Wave

The refraction fan diagram is a particularly valuable tool for determining the wave to be used in sea wall design. This technique consists of projecting a number of orthogonals seaward from the shore point at which wave data is required. These orthogonals, plotted 5 or 10 degrees apart, make up a set of refraction fans (for each of the various wave periods which may be anticipated at the site). A period increment of 2 seconds is usually used between each set. These orthogonals determine the deepwater wave directions corresponding to the near shore directions of wave approach. After eliminating the directions of deepwater wave approach

which are covered by island screens and mainland coast headlands, companion orthogonals are drawn on each side of the remaining initial direction orthogonals from seaward toward the shore line. From these, refraction coefficients for each deep water wave direction are determined. Applying at the refraction coefficients to the deep water wave data as obtained by either the Sverdrup-Munk-Bretschneider method or the wave spectra method, the wave characteristics at the sea wall location are determined (Ref. 4, p. 29). Tabulating refracted wave heights at the shoreline for the various anticipated deep water wave heights and periods permits immediate selection of the wave which will produce the greatest wave height at the structure's location. This is then the design wave for use in wave pressure and run-up calculations.

It must be borne in mind that refraction diagrams give accurate results only where the bottom hydrography is relatively simple and consists of fairly flat slopes (less than 1 on 10). Where other more complex hydrography prevails, wave observations over a long period of time may be the only method of selecting the design wave. Such data should be tempered with as much historical wave data as can be accumulated.

G. Energy Dissipation

Having followed the ocean wave from its point of generation, through a decay distance, across shoaling topography to the breaker line and being able to predict its characteristics at that point, it is now possible to discuss the key to successful shore protection design insofar as wave energy is concerned. This key is the knowledge of the manner in which wave energy is dissipated.

Basically the wave loses kinetic energy (1) through an increase

in its potential energy, i.e., wave run-up, (2) through generation of force against shore structures or reflection from a natural coast line, and (3) through heat generation. This last method of energy dissipation has little practical importance and will not be discussed further.

(1) Run-Up. Some wave energy is dissipated by turbulence as a wave breaks. The remaining energy is transformed into potential energy as the wave runs up the slope. A surging wave does not experience this loss of energy through turbulence; however, it is more subject to reflection and must be analyzed in a different manner. A third situation in the run-up problem occurs when the wave breaks seaward of a structure and run-up must be computed for the composite slope involved. It can be appreciated from the above that the location of the breaking point and the nature of the wave are important factors in determining the run-up on a coastal structure.

Run-up, the vertical height to which water will rise on a given structure, determines the required structural crest height to prevent overtopping and resultant flooding on the land side. This is a critical factor in design since overtopping not only results in property damage shoreward of the sea wall but also may cause complete destruction of the protective structure itself through back face erosion, excess water pressure and seepage. Past practice in this field featured the arbitrary use of a ratio of run-up to wave height of 1.5 or where a more sophisticated approach was deemed necessary, a ratio of 8 tan θ was used. (θ = slope of structure.)

In 1952, the Beach Erosion Board initiated a series of tests on wave run-up and overtopping (Ref. 22). Profiles of various conventional shore structures and of relatively gentle slopes corresponding to those

obtained with artificial protective fills were tested. All tested structures were fronted by a 1 on 10 slope and were of impermeable construction. The following conclusions were reached from these tests:

(1) Run-up increases with the increased depth at the toe of a shore structure, d, until a depth-wave height ratio of between 1 and 3 is reached. Beyond this value the run-up decreases slightly.

(2) For structures with deep water at the toe ($d/H_o \geq 3$), run-up increases as the slope steepens. A vertical wall, however, resulted in lesser run-up than the slopes exceeding about 1 on 4 except where the structure's toe was at zero depth. Curved walls exhibited the highest run-up values, all other factors being equal. Adding recurvature was found to decrease the run-up materially.

(3) For structures with the toe at zero depth, run-up increased with slope up to about 1 on 3 and then remained practically constant.

Figure 8 summarizes the results of the Beach Erosion Board Tests and illustrates graphically that an R/H ratio of 1.5 is not valid for many types of structure particularly where waves are encountered of steepness less than that corresponding to an H'_o / T_2 value of 0.3. The quantity H'_o/T^2 is directly proportional to H'_o/L_o and was adopted as a parameter for the tests because of its ease of use. When using these curves, it must be noted that a foreslope steeper than 1 on 10 will generally increase the run-up although where d is several times the wave height, this slope becomes of less importance. Further, these curves are for smooth, impermeable structures.

Concurrently with the above tests, Granthem was conducting an investigation at the University of California to determine the effects of

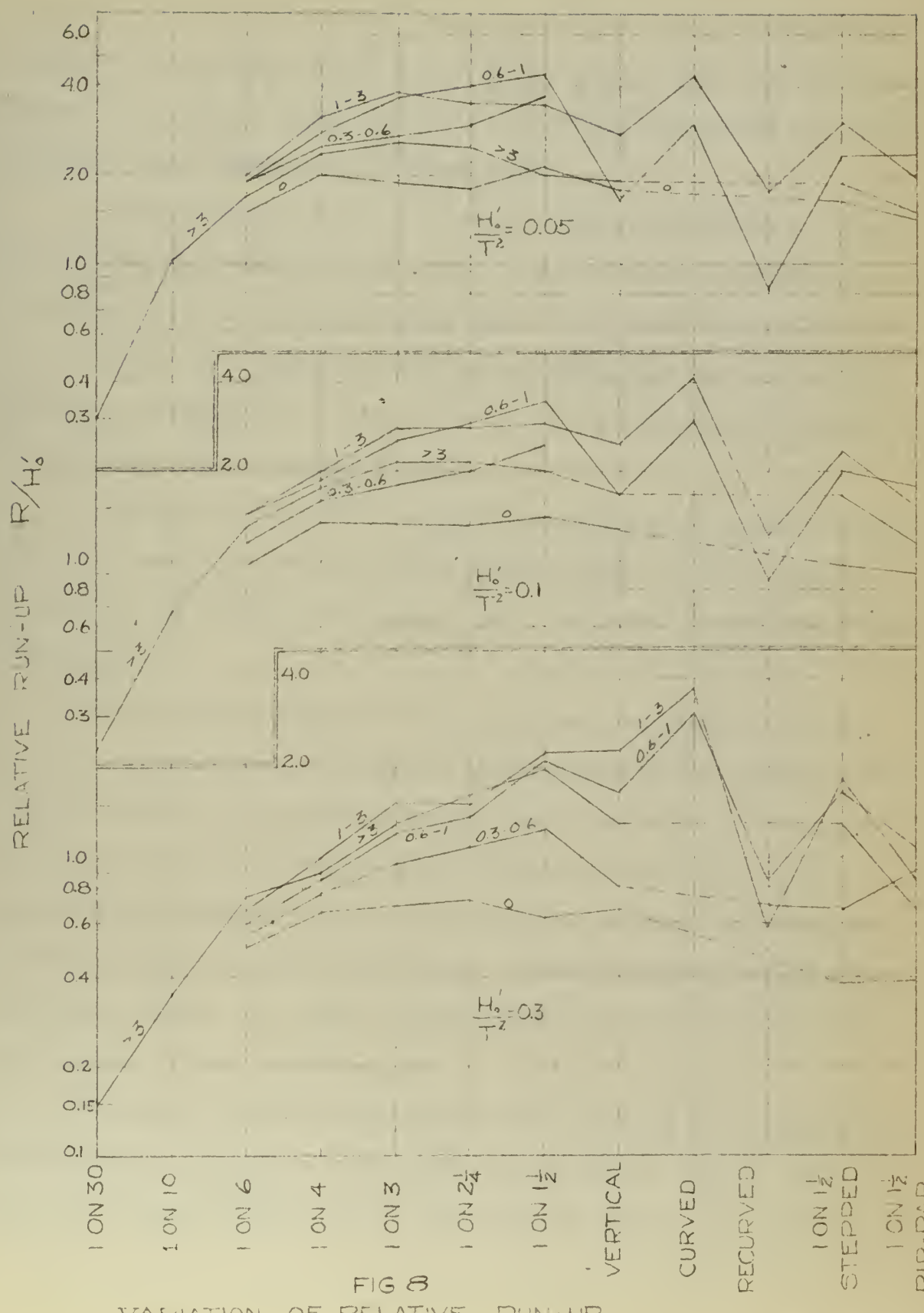


FIG 8

VARIATION OF RELATIVE RUN-UP (after Saville 1956)

slope and porosity on run-up. He found that maximum run-up occurred for slopes of about 30° which invalidated some of the "rule of thumb" methods being used in practice at that time. It was further noted that increasing porosity had a great reducing effect on the run-up experienced for constant values of wave steepness and structure slope.

In an attempt to arrive at factors which could be used to compute the effects of permeability and roughness on wave run-up, additional tests were undertaken by the Beach Erosion Board. (Ref. 21) Slopes from 1 on 30 to vertical were tested with the water depth at the toe held constant since the previous tests had shown that (d) had a negligible effect when it was several times the wave height.

The quantities H'_oT^2/d^2 and H'_oT^2/k were used as parameters for the effects of roughness and permeability respectively. In these factors, d is equal to the median grain diameter and k equals the permeability of the slope material.

Using these parameters, it was shown that the effect of slope roughness on wave run-up increases as the parameter H_oT^2/d^2 decreases; for constant values of the parameter and the slope, the effect of roughness increases as the wave steepness decreases; for constant values of the parameter and steepness, the effect of roughness decreases as the slope steepens. These same relationships were observed using the permeability parameter; however, the effect of permeability on run-up was found to be more pronounced than roughness alone.

In normal protective construction both roughness and permeability effects must be considered in computing the anticipated run-up. Taking these two factors together, the following general results can be observed:

(a) The reduction in run-up on a rough and permeable slope as compared with a smooth slope decreases as wave steepness increases and as the slope steepens.

(b) For very steep waves ($H'_o/T^2 > 0.25$), relative run-up is greatest for a slope of 1 to 2. For waves of lower steepness, the critical slope is approximately 1:5.

Curves showing the relative effect of the roughness and permeability variables for various slopes are contained in Ref. 21. These curves are only valid for water depth at the toe of the structure exceeding 3 times the wave height ($d/H'_o \leq 3$). As depth at the toe decreases, the relative run-up increases to a value which may be twice that predicted by the test data. The curves are valid for many of the situations encountered in practice but should not be used for rubble mound structures where the median material diameter approaches or exceeds the wave height, H.

Utilizing data from the above mentioned tests, Hunt has developed some general run-up formulae which are of practical value to the coastal engineer particularly when the situation under investigation is not covered by one of the test curves. Hunt considered first a wave having ($(H_i/T^2)^{1/2} <$ tan α) where α is the slope of the sea wall. In water of sufficient depth these waves will break on the structure and the following empirical formula is valid:

$$\frac{R}{H} = \frac{2.3 \tan \alpha}{(H/T^2)^{1/2}}$$

When $(H/T^2)^{1/2}$ exceeds tan α, the incoming waves will surge up the slope and the run-up increases from 2.3H for $(H/T^2)^{1/2}$ = tan α to a maximum value of 3.0H. As indicated previously, there is a critical slope which will

result in maximum run-up for various wave steepness ratios. In general for storm waves the run-up value of 3.0H should be used for slopes greater than or equal to 45°. For reduced slopes the above equation applies.

Hunt corrects for roughness and permeability by adding the terms (r) and (p) respectively to the general equation. Values for these factors corresponding to a slope of 1:10 are tabulated in Ref. 13 and indicate that run-up may be reduced as much as 62%. The reduction will, in general, be greater for flatter slopes and the reader is referred to Hunt's work for curves giving the observed effect of these variables.

Composite slopes and berms as shown in Fig. 9 are frequently used in coastal structures to reduce space and material requirements. The lower slope is designed so that all incoming waves will break on it. The wave rush up is then dissipated on the second slope. To achieve this, the initial slope is made less than $(H/T^2)^{1/2}$ and the break in slope is located at the highest anticipated water level. With this arrangement, the R/H vs. H/T^2 curve will fall between the curves for the two slopes involved and will be approximately parallel to them. Extending the general formula to cover this situation Hunt obtained

$$\frac{R}{H} = \frac{2.3}{(H/T^2)^{1/2}} \frac{(\tan \alpha_1 + \tan \alpha_2)}{2} (s)$$

The factor (s) corrects for additional turbulence induced by the composite slope and varies with $(\alpha_1 - \alpha_2)$ as follows:

α_1 = 1:3, α_2 = 1:6, s = 0.9

α_1 = 1:3, α_2 = 1:10, s = 0.8

α_1 = 1:10, α_2 = 1:3, s = 0.8

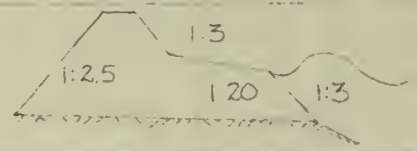
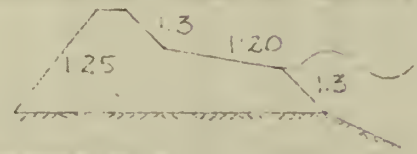
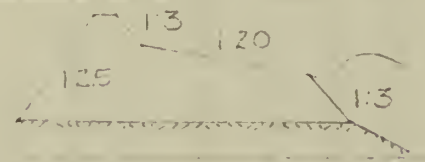
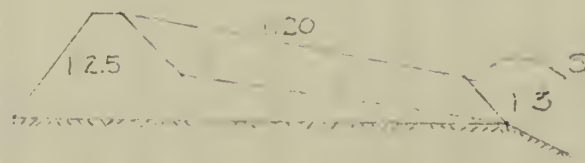
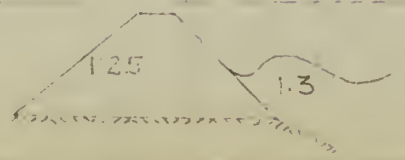
STRUCTURE	SKETCH		WAVE DATA H (FT.)	WAVE DATA T (SEC.)	$\frac{H}{T^2}$	$\frac{R}{H}$	R FT. ABOVE SWL	CREST ELEV.	WIDTH	AREA ($FT.^2$)
30 FT BERM		SWL	12	6	.333	0.89	10.7	15.7	122.8	997
50 FT BERM		SWL	12	6	.333	0.72	8.6	13.6	128.3	924
70 FT BERM		SWL	12	6	.333	0.63	7.6	12.6	139.8	977
COMPOSITE		SWL	12	6	.333	0.61	7.3	12.3	202.8 (1:3 TO 1:20) 163.7 (1:20 TO 1:3)	1625 764
CONTINUOUS SLOPE		SWL	12	6	.333	1.05	12.6	17.6	107.8	1045

FIG 9
COMPARISON OF RUBBLE MOUND PROFILES
(after Hunt 1958)

The fact that these last two combinations give the same run-up reduction factor is of great practical importance because where adequate space is available, great material savings can be achieved by utilizing the 1:10 slope outboard. This is illustrated in Fig. 9.

At times it is impractical to extend the second slope of a composite structure far enough to dissipate all run-up. In that case, a third slope or berm is utilized. Tests at the Waterways Experiment Station, Vicksburg, have shown that run-up is reduced as B/L increases where B = berm width and L = wave length. As a general rule, B/L should be greater than 1/5. The slope of the upper surface is not of great importance although steeper slopes of 1:3 or 1:2 result in greater turbulence and therefore reduced run-up. As with the normal composite structure, the break in slope at the berm toe should be at or slightly above the highest anticipated water level.

(2) Wave Overtopping. Economics has a pronounced effect on all engineering works. This statement is equally true for protective structures to the extent that frequently it is not feasible to construct a sea wall or bulkhead of sufficient height to prevent all overtopping. In that case, it is essential that the engineer have some knowledge of the amount of water which will pass over the structure in order that adequate drainage and pumping facilities may be installed to prevent flooding and to safeguard the protective works. The most reliable data on this problem was developed by Beach Erosion Board tests and is available in Refs. 4 and 7. In using these data it should be noted that the structures tested were impermeable in all cases and smooth with the exception of one riprap slope. Also, uniform waves were used in the tests which obviously do not occur in nature. It is therefore necessary to compute a combined overtopping value based

on the various wave heights in the wave spectrum. A method of accomplishing this is presented in Ref. 4.

(3) Wave Forces on Impermeable Structures. In considering the forces exerted by the sea against obstructions such as coastal protective structures, three distinct conditions can occur. These are as follows:

(a) The approaching wave does not break but forms a standing wave or clapotis which has a crest height above still water level greater than the height of the impinging wave.

(b) The wave breaks directly on the structure.

(c) The wave breaks seaward of the structure and progresses up the slope as a turbulent mass of water.

In the first of these categories, the pressures are hydrostatic while in cases (b) and (c), dynamic effects result in pressures which may be many times the hydrostatic pressure. Hunt, in his work "The Design of Sea walls and Breakwaters" has recommended that reflection of the wave be minimized to reduce surging in harbors and to prevent erosion of the foundation material at the toe of the structure. It was found that the expression for critical slope

$$i = \sqrt{\frac{H}{T^2}}$$

as developed by Iribarren and Nogales corresponded to the actual critical slopes observed in tests by Caldwell, Granthem, and the Beach Erosion Board. Hunt therefore recommends that the slope of the structure be less than $\sqrt{\frac{H}{T^2}}$ for the design waves to insure breaking and energy dissipation through turbulence.

While the results desired in causing the wave to break are important, it must be noted that adherence to this recommendation will cause extremely

high pressures on the structure which may necessitate great expenditures to insure stability.

Case (a)

The determination of the breaking depth of wave has been previously discussed. In general, if the depth at the structure is greater than 1.3 times the wave height, breaking will not occur and a clapotis will be formed. The method developed by Sainflou for determining the pressures exerted by an unbroken wave against a vertical structure is in wide use. Minikin has correlated this method with the results of various full-scale tests, including those of Cagli on the Genoa breakwater, to arrive at simplified expressions for these pressures. Basically, his proposed method is as follows:

(a) The zero point on the pressure diagram should be at 1.66 H above still water level where H is the deep water wave height of the design wave.

(b) Maximum intensity of pressure occurs at still water level and is equal to wH (hydrostatic pressure due to a wave of height H in the open sea).

(c) Pressure increases uniformly from the zero point to the still water level.

(d) Below still water level pressure is assumed constant and equal to wH.

The resulting pressure diagram compared with Sainflou's and Cagli's is shown in Fig. 10 for a wave of 16' height, and 240' length in water depth of 33'. This pressure diagram redrawn in Fig. 11 is based on a water level

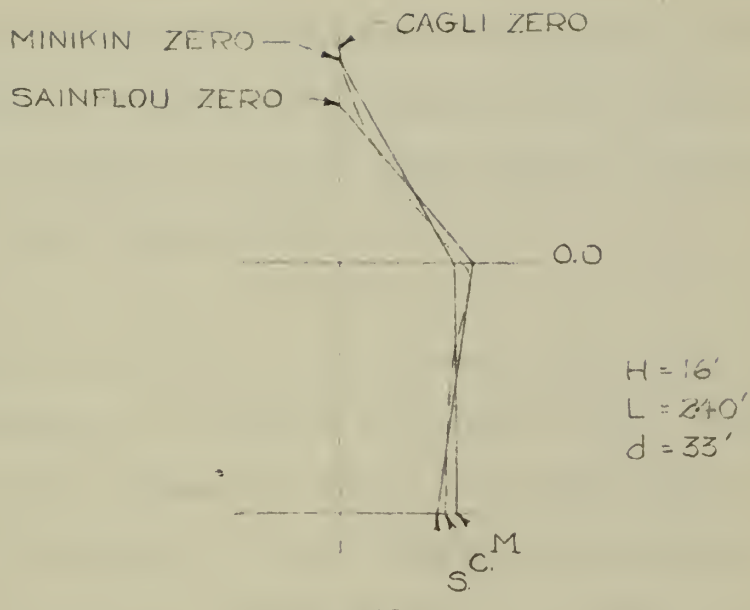


FIG 10

COMPARISON OF MINIKIN, CAGLI, AND SAINFLOU PRESSURE DIAGRAMS FOR UNBROKEN WAVE

(after Minikin 1950)

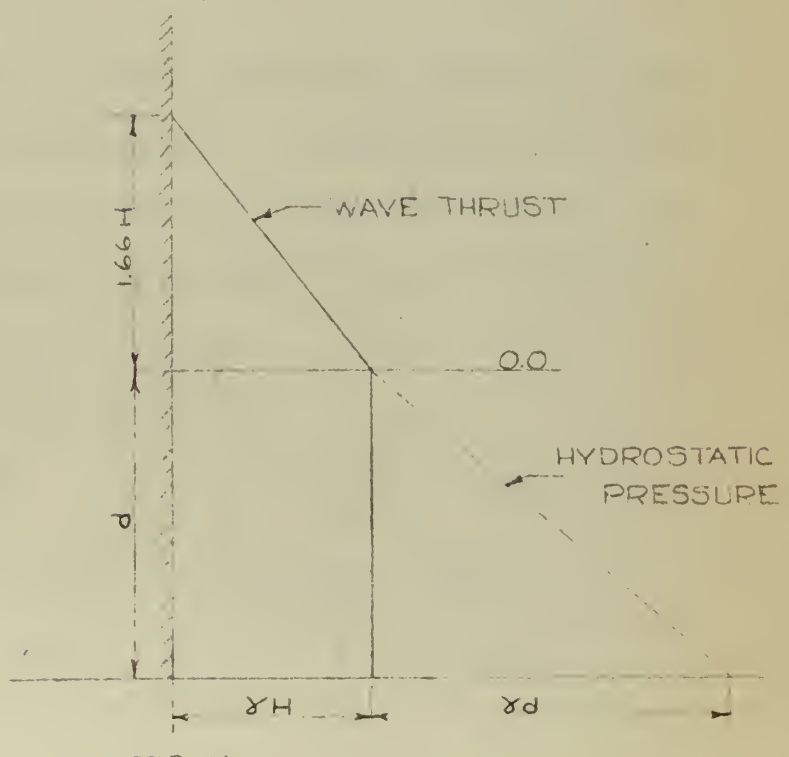


FIG 11

UNBROKEN WAVE PRESSURE - MINIKIN METHOD

behind the wall equal to still water level to seaward. Should no water pressure exist on the landward side of the structure total hydrostatic pressures as shown by the dashed lines must be considered. To handle the various intermediate situations, it is recommended that this full hydrostatic pressure be assumed in all cases compensating for the pressure to landward with a normal pressure diagram.

Case (b)

The mechanism resulting in shock pressures when a wave breaks directly on a structure was discovered by Baynold in 1939. Through a series of model tests, he found that when a wave breaks against a structure, a pocket of air is trapped by the crest being projected forward. This air pocket is then compressed by further movement of the water until a virtual explosion occurs hurling the water upward in an aerated condition. When the width of this air pocket is small in proportion to its height, high shock pressures will occur.

Based on these tests by Bagnold, Minikin has developed a useable approach to the determination of pressures exerted by waves breaking on a structure. The total pressure is due to a combination of dynamic and hydrostatic pressures. These are given respectively by the following formulas:

$$P_m = \frac{101\ H_b w}{L_o} \times \frac{d}{D}\ (D + d)$$

$$P_s = \frac{wH_b}{2}$$

where H_b = height of wave breaking on the structure (feet)

w = unit weight of water (lb. per cu.ft.)

d = depth of water at the structure (feet)

D and L_o = deeper water depth and wave length (feet) usually taken one wave length from the structure

The sum of these two pressures gives the peak pressure which occurs at still water level. Above and below this level the dynamic pressure reduces parabolically to zero at distances of $H_b/2$ and $-H_b/2$ respectively. The hydrostatic pressure diagram is naturally triangular having its apex at $H_b/2$ above still water level. The resulting diagram is shown in Fig. 12.

Case (c)

This case is frequently encountered in shore protective structures which are fronted by a beach. Under storm conditions or as a result of erosion the beach may become submerged and case (b) or even case (a) may apply. To date no model studies or full scale tests have been applied to this problem and theoretical relationships must be utilized.

The Beach Erosion Board recommends the following procedures in this case. The water mass in a wave after breaking is assumed to move forward with the velocity of the wave just prior to breaking. This turbulent mass of water moves up to and over the shoreline until the protective structure is encountered. For a structure seaward of the shoreline at still water level, the dynamic pressure is assumed to be evenly distributed between still water level and a height, h_c, equal to 0.7 H_b since model tests have shown that upon breaking, approximately 70% of the full wave height is above still water level. This dynamic pressure is given by $P_m = \frac{wC^2}{2g} = \frac{wd_b}{2}$ where w = unit weight of water.

For a structure shoreward of the shoreline, the velocity and height of the water mass is dependent on the run-up which the wave will experience.

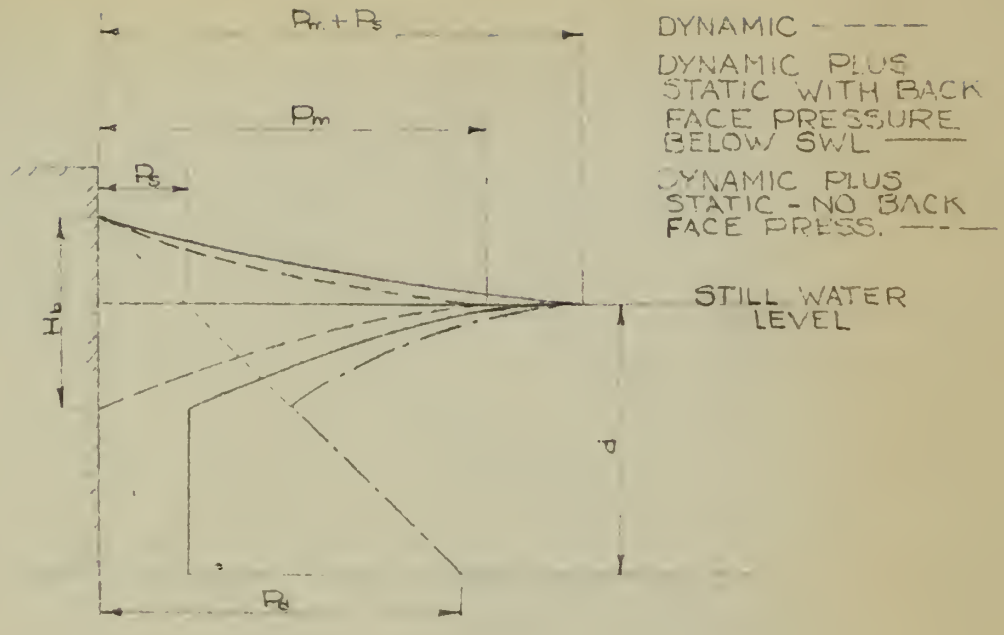


FIG 12
BREAKING WAVE PRESSURE DIAGRAM

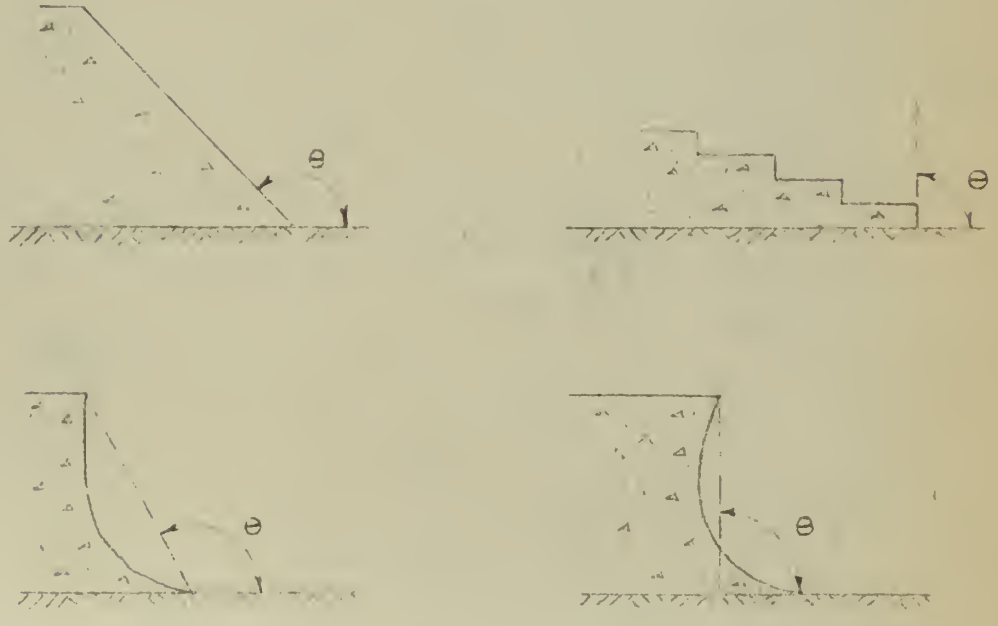


FIG 13
DETERMINATION OF θ FOR VARIOUS SEAWALL SHAPES (Ref 4)

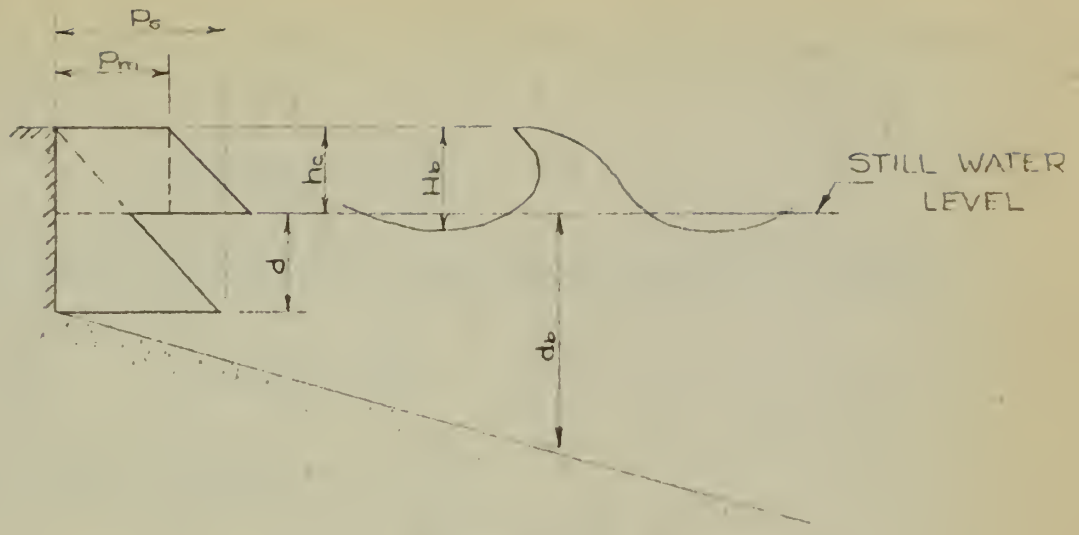


FIG 14A
PRESSURES FROM BROKEN WAVES
WALL SEAWARD OF SHORE LINE (Ref. 4)

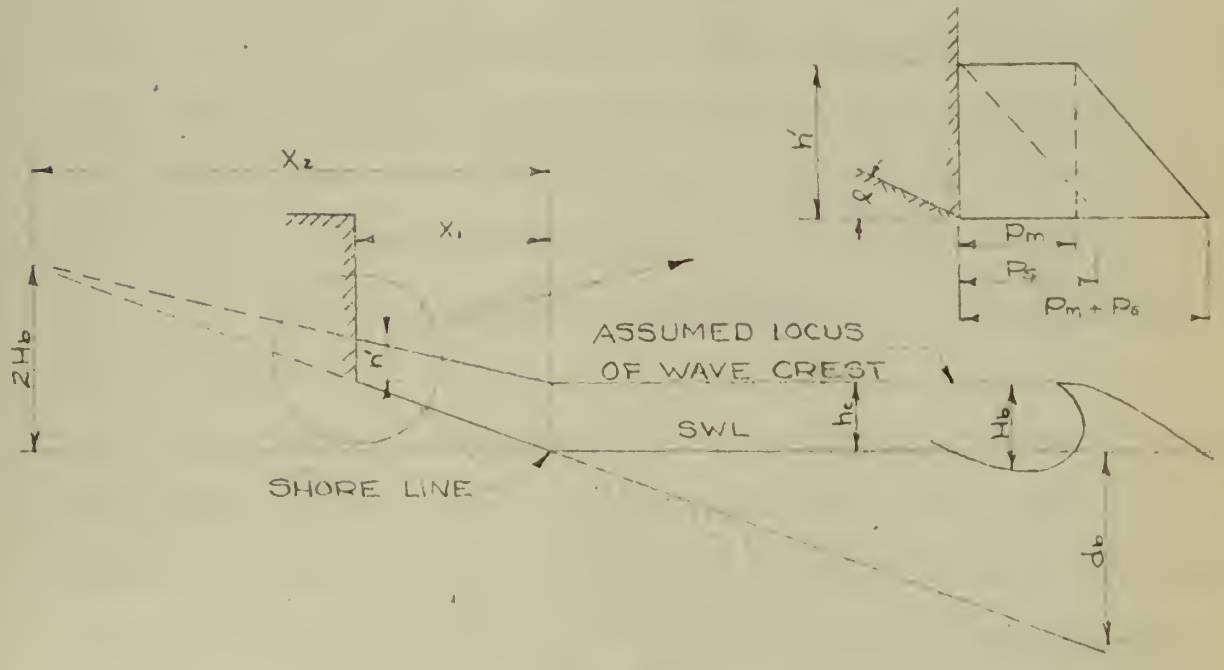


FIG 14B
PRESSURES FROM BROKEN WAVES
WALL SHOREWARD OF S.W.L. (Ref 4)

Referring to Fig. 14B, it can be seen that the height can be approximated by

$$h' = h_c(1 - \frac{x_1}{x_2})$$

assuming that the velocity is reduced in the same ratio we obtain

$$v' = C(1 - \frac{x_1}{x_2})$$

and the dynamic pressure is therefore

$$P_m = \frac{wv'^2}{2g} = \frac{wd_b}{2}(1 - \frac{x_1}{x_2})^2$$

In both locations, hydrostatic pressure corresponding to the height of run-up on the structure must be added to the dynamic pressures indicated above. Pressure diagrams as shown in Fig. 14 will result. This simplified approach will provide approximations which are believed to be conservative. For a more accurate determination, it is recommended that the data in the section of this work on wave run-up be utilized to determine h' for structures landward of the shoreline whose run-up characteristics are known.

The formulae developed for waves breaking on or seaward of coastal structures apply for cases of essentially vertical walls. Where the wall slopes back at an angle θ, the horizontal component of the dynamic pressure should be reduced to $P'_m = P_m \cos^2\theta$. Forces on stepped-face structures are computed as if the face were vertical. The determination of angle θ for other typical sea walls is shown in Fig. 13.

In all of the above cases, where the wall is lower than the maximum height of the anticipated pressure diagram, draw the diagram as though the

wall were higher and disregard all pressure above the actual wall crest.

(4) Wave Forces on Rubble Mound Structures. Rubble mound structures are frequently used in coastal protection. In general, they are less expensive than an equivalent gravity wall where an adequate supply of rock is available, and they never fail completely under wave attack in the catastrophic manner associated with other types of shore protective structures. The qualities of great permeability and roughness inherent in a rubble mound structure aid in the dissipation of wave energy and thus reduce run-up and overtopping.

The destructive action of waves on this type of structure is characterized by the lifting of individual stones until they are finally dislodged and tumble down the slope under the force of gravity. In 1938, Iribarren presented a formula for the required weights of individual stones above the water surface of a rubble mound structure. (Ref. 14) The formula was based on a force diagram for a single stone considering the coefficient of friction equal to 1. A coefficient, K, was evaluated by observations of two existing rubble mound structures. In 1955, Hudson proposed several changes to the original Iribarren formula to make it dimensionally correct and to include a coefficient of friction greater than 1. The revised formula is as follows:

$$W = \frac{K' \gamma_r \gamma_f^3 \mu^3 H^3}{(\gamma_r - \gamma_f)^3 (\mu \cos \alpha - \sin \alpha)^3}$$

where W = weight of individual cap rock

K' = dimensionless coefficient

γ_r = specific weight of rock

process. As indicated in Chapter II, the principle of mass transport results in the wave having the ability to transport materials shoreward. When the wave breaks and its motion is converted into uprush, particles in suspension are carried toward the beach crest. This particle motion is opposed by gravity and by the ensuing return flow or back rush. Under the action of these forces, combined with any longshore current, a particle is moved along the foreshore in a saw-toothed path. This material is called "beach drift." Other materials, thrown into suspension by the turbulence of the breakers, are moved principally by the littoral currents. Still further movement is evident in those materials which never actually go into suspension but are rolled and slid along the sea bottom. Basically, the coarser materials, whatever their mode of transport are moving shoreward and the finer sediments are carried seaward for eventual deposition in the intermediate or passive zones.

(E) Particle Sorting

In discussing the transport of littoral materials, it was evident that extensive natural particle sorting was occurring. This sorting action is largely a function of the particle's settling velocity which is determined by size, shape and density. Observations have confirmed that median grain size is a satisfactory parameter for general evaluation of transportability. It has been shown that for a given depth, slope, and wave, there is a characteristic material which is shifted back and forth by the oscillating wave currents but does not actually leave the orbit. This material is defined as an "equilibrium particle size." This equilibrium particle size increases as depth decreases since water velocity is increased assuming

constant slope and wave characteristics. For constant wave characteristics and depth, size increases as slope steepens because gravity effects become more pronounced. The equilibrium size for constant depth and slope also increases as the wave height or period decreases since less bottom velocity will be available to cause motion. The key point in explaining the material sorting process by utilizing the concept of equilibrium particle size is that all particle sizes at any point larger or smaller than the equilibrium size are in transit shoreward or seaward respectively. Following this reasoning, it is apparent that littoral materials move generally within a depth range compatible with their size or resistance to transport. The coarsest material is usually found in the surf zone just shoreward of the plunge point of the impinging waves. In addition to the transverse sorting process described above, a progressive decrease in the median grain size with distance from the source can sometimes be observed within the physiographic unit.

(F) Beach Profiles

A stable shoreline is one in which the supply of material to the unit equals the losses from the unit. Should the supply exceed the losses, the shoreline will build up or accrete and the beach can exhibit one of the following profiles:

(1) Over Nourished. In this case the material balance exceeds the quantity which the waves can shape into the so-called "equilibrium profile." The beach will have a very irregular form showing offshore bars and shoals.

(2) Sufficiently Nourished. In this case the waves can shape the balance of materials into the equilibrium profile.

A third type of beach profile occurs when the supply is less than the material loss. The beach is said to be "under nourished," erosion occurs and the coastline recedes. It should be noted that an equilibrium profile is maintained as the beach is eroded.

The term equilibrium profile as used above is defined as the stable profile of maximum steepness apart from seasonal fluctuations. This profile is affected by various factors including the following:

(1) Characteristics of the waves

(2) Quantity and character of the littoral materials

(3) Changes in ocean level

(4) Ground water level

(5) Any feature which results in excessive turbulence such as a sea wall or rock outcrop

To investigate the interaction of the above variables, the Beach Erosion Board conducted a series of laboratory tests from which the general process of profile shaping can be deduced (Ref. 31). It was found that ripples first formed seaward of the breaker line and that an offshore bar built up at the breaker line. The ripples moved shoreward and lost their identity at the seaward face of the bar. The bar also moved shoreward and flattened while the water depth at the toe remained constant. Flat waves continued to move material shoreward raising the berm crest and filling in behind the bar which eventually took on the form of the characteristic steep step at the seaward toe of the foreshore. Steep waves tended to move material offshore but the step remained and the foreshore was gradually flattened. It was observed that for a given wave steepness, foreshore slopes become steeper as grain size increases. With increasing wave

steepness, however, the foreshore slope flattens. The slope above still water level appeared more dependent on grain size and less on wave steepness than is true of the foreshore slope in general. The slope of the offshore section was found to be a function of wave steepness, particle coarseness and the original slope of the bed.

In the past there has been a tendency to overemphasize the relationship between beach slope and particle size. Actually this situation is more of an effect than a cause. Consider the typical winter conditions of large waves and a high water table. The heavy turbulence and upward seepage forces combine to throw the fine materials into suspension and a plane or concave upward profile of predominantly coarse material results. In the summer season, lower waves and lower ground water level favor accretion This process continues to increase the beach slope until the backwash removes material as fast as it is deposited. Since the run-up energy of a wave is reduced by beach friction and percolation, the coarase materials are not moved back down the berm as readily as the fines. The result is a profile of relatively coarse material which is convex upward. From the above, it is apparent that the slope of the foreshore is determined primarily by the character of the waves. The grain size on the other hand is dependent on both wave characteristics and foreshore slope. Other factors being equal, a steeper slope will increase the velocity of backwash and the amount of turbulence. This causes an increase in the maximum grain size in suspension and more fines are removed; therefore, the mean grain size remaining in place on the berm is increased. This discussion explains the frequently observed characteristic of a steep beach with a coarse grained material. Figure 16 illustrates the terms used in discussing beach profiles.

FIG 16
BEACH PROFILE (Ref 4)

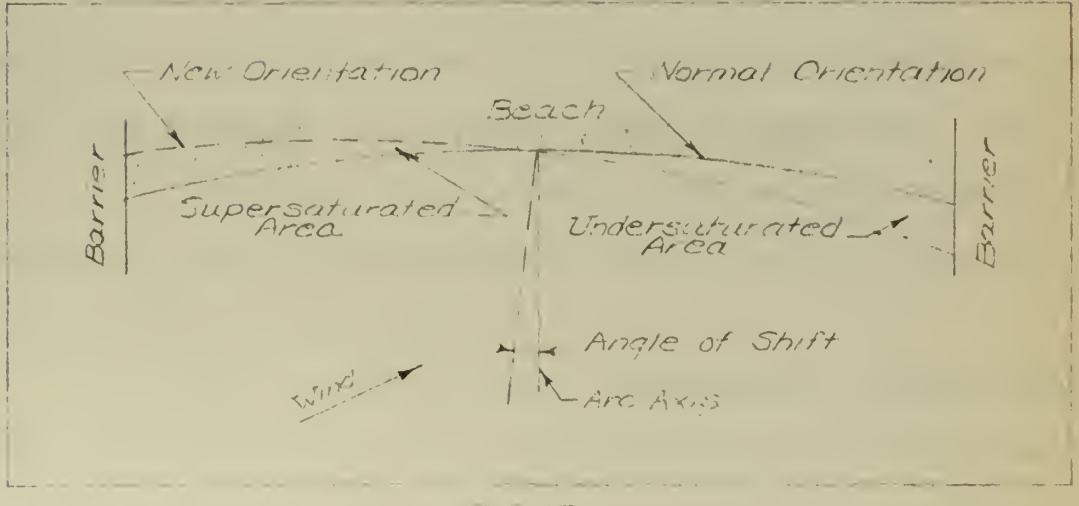


FIG 17
REORIENTATION OF ARC OF EQUILIBRIUM (Ref 35)

(G) Shoreline Movement

As indicated above, once created, the equilibrium profile maintains its form as the shoreline advances or recedes unless an excess of littoral material becomes available. This latter situation would be only temporary and the shoreline will continue to change position until equilibrium is established. Since the forces acting are constantly changing, the shoreline although basically stable is in constant movement. This movement takes the following forms:

(1) Migrating Waves. The shoreline is never straight but is instead composed of many small curves or waves. These are related to the cell-like current action previously described. Each curve may have an arc length of from 300 yards to 2000 yards and an amplitude of 60 to 80 yards on unprotected coasts. These forms move along the beach at rates up to one mile per year. A similar phenomena occurs when a pool is left on the beach at low tide. The exit flow from this pool will move gradually along the beach in the direction of the prevailing wind and flood tide.

(2) Seasonal Fluctuations. As indicated in the discussion of beach profile, the shoreline generally recedes in the winter and builds up in the summer primarily as a function of wave and ground water level changes.

(3) Variations in Material Supply. This process of advance when the supply is greater than the loss and recession under the opposite situation has previously been outlined.

(4) Changes in Beach Arc Axis. Observations on various natural beaches have shown that an "arc of equilibrium" tends to form between headlands or other barriers which confine the beach materials. Both the bar-

riers and the natural curvature appear to be essential for over-all beach stability. This natural curvature approximates an arc of a circle subtending an angle of approximately 0.25 radians. The orientation of the arc axis is based upon the predominant climatic conditions.

A basic requirement for beach stability is that the waves must strike normal to the shoreline; otherwise, drift occurs as nature attempts to correct the beach alignment. Wave refraction is therefore a controlling feature of beach stability. If the direction of wave approach were constant, the arc of equilibrium would eventually assume a shape such that wave approach would be normal to the beach in all locations. Unfortunately, this ideal situation can never occur since the direction of ocean wave approach changes and local winds and tides modify the normal refraction pattern. For any beach, however, a long term or average orientation will occur based upon the cumulative effect of all wind and wave directions.

Figure 17 shows the normal orientation of the arc of equilibrium between two barriers. A storm with strong cross winds as shown will cause a shift in the arc axis. The triangular area to windward between the original arc and the reoriented arc becomes unstable and erodes while the corresponding area to leeward experiences accretion. This process continues until the new arc of equilibrium is established. Reorientation of the arc is a continual process and involves constant transfer of beach material from one end of the arc to the other. In a long beach, this is a major readjustment and completion would only be possible as a result of a seasonal or major climate change. The transfer is very apparent on shorter beaches such as between groins and may be completed in a relatively short time.

Beach length is then the controlling factor in the degree of axis shift. From observations, this shift can vary from 10° for a 200' beach to 2° or less for beaches of length exceeding 4000 feet.

The theory of the "equilibrium arc" is a useful tool in understanding the mechanics of beach stability. The effect of any artificial barrier can most easily be forecast by this means.

(H) Direction and Quantity of Littoral Transport

The mechanics of littoral transport and deposition have been outlined but the engineer's problem of determining the predominant direction and quantity of this transport remains to be solved. In general, the solution depends on the application of various basic relationships to the particular situation encountered. The effects of existing structures provide the most reliable information and generally outweigh all other evidence. The condition observed at small structures such as groins can be assumed to indicate the direction of littoral transport during the immediately preceding period. To avoid misinterpretation of the seasonal effects, observations should be made over at least a one-year period. Breakwaters and entrance jetties involve large enough quantities of material that the situation at any time can be considered indicative of the over-all direction of littoral transport. Prominent headlands do not present as clear an indication since these features may actually constitute boundaries of the physiographic unit. In general, wave cut cliffs with no beach usually indicate a down-drift shore while wide, well developed, beaches are found in the up-drift area. The location and formation of tidal inlets may indicate the predominant direction of littoral drift since they tend to

migrate in the same direction. This evidence should be used with caution, however, since reversals and recent break throughs can completely confuse the picture.

Attempts have been made to relate the directional components of wave energy to the direction of littoral transport. This method has proven reliable on uniform coasts but is subject to many inconsistencies where a complex topography exists. Its use for general practice without extensive verifying observations of the type heretofore indicated does not appear warranted.

Direct measurement of the littoral current utilizing floats outside the breaker line and dye within that area will sometimes give an indication of direction. Observations must be conducted over a period of at least one year to be of any value.

The rate of littoral transport can only be measured accurately by determining the actual amount of accretion or erosion at a substantially complete artificial barrier during a known period of time. To compensate for seasonal changes, surveys should be taken at the same time each year. When this method is to be used, the survey must cover the entire area of accretion and/or erosion which will be affected during the period of the study.

When important coastal structures are planned, the use of a model study is recommended. This approach can give important information on wave effects and with the potential supply of littoral materials known, a reliable picture of transportation and deposition of beach material can be determined.

(I) Rates of Material Supply and Loss

The sources of littoral materials have been discussed previously.

Losses from a specific beach area include the following:

(1) Movement of material laterally out of the area by one of the mechanisms of littoral transport. This loss is measured directly as previously described or may be estimated by considering rates of transport at points above and below the area in question.

(2) Movement of fine materials in suspension in an offshore direction toward final deposition in the intermediate or passive zones. The quantity of material lost through this process can not be accurately determined at this time. Work is being done in this field, however, by the Beach Erosion Board.

(3) Loss of material shoreward by wind action. The amount of material lost through this process is difficult to ascertain but can best be estimated by dune surveys.

(4) Loss of material into near shore submarine canyons. This is an extremely important loss where it occurs and can only be determined by defined comparative surveys of the area.

(5) Abrasion. In the past the abrasion of beach materials and subsequent transport of the fines for deposition in the depths of the sea have constituted a favorite geological theory. Recent observations have shown that this is actually a loss of very minor importance.

(J) Conclusion

The information presented in this chapter is at best a brief outline of the mechanics of littoral processes. An understanding of these basic relationships is essential for the coastal engineer since a structure extending into the sea at any tide condition will affect and be affected by the natural littoral regimen. Failure to observe and allow for these

processes in the design of a coastal structure may result in the structure's becoming the author of its own destruction and in addition may cause great property damage to adjacent coastal areas. In the same manner, removal of beach or sea bed materials for use as aggregate or fill will unbalance the delicate equilibrium, frequently with serious results as the sea attempts to reëstablish the stability pattern.

For detailed information concerning laboratory tests, field observations, and general theory, the reader is referred to the many excellent publications of the Beach Erosion Board.

PART II

CHAPTER IV

Application of Basic Data to Sea Wall Design

A. Functions of Sea Walls

A stable beach backed with sand dunes is the ideal form of coastal protection against storm seas as well as against normal wave action. Unfortunately in many areas, dunes have been removed to permit real estate development and natural littoral processes have been altered by groin and jetty construction. In other cases, previously stable beaches have begun to erode as a result of protective works' cutting off the supply of littoral materials from eroding headlands. Although artificial nourishment of eroding beaches, construction of groin systems, and sand by-passing at tidal inlets have been used with success in many areas, loss of adequate beach width and dune removal have resulted in vulnerability of shore properties to the effects of storm seas. A sea wall is essential to provide protection in this case where the property behind the beach is highly developed and there is a probability of severe storm attack.

Many of the world's coastlines have never had an adequate supply of littoral materials or the favorable conditions necessary to promote the formation of stable beaches. Erodible headlands fall within this category. Towns, transportation arteries and industries located on these headlands undoubtedly appeared safe from the sea at the time of their construction. Many of these are now menaced by the rapid recession of the blufflines and construction of some type of sea wall to prevent further erosion is imperative.

The occurrence of low-lying agricultural lands adjacent to the seacoast is not limited to the Netherlands but is found in many of the world's unglaciated lowland areas. These areas with their communities and farms must be protected from flooding by the sea under storm conditions. Where inundation is inevitable, protection must still be provided against direct wave action by the construction of sea walls.

In each of the above situations, the following general function of a sea wall is evident: A sea wall fixes the high water level along a particular alignment allowing no further recession of the shoreline and protecting the inshore area from destructive wave attack. It must be noted that the sea wall protects only that area immediately shoreward from it and may cause increased erosion to seaward and at both ends of the protected length. This characteristic if not considered in design, may cause the protective structure to become the author of its own destruction as a result of scour at the toe or flanking action at the ends.

B. Classification by Type

Sea walls can be classified structurally as follows:

(a) Gravity walls. Best located above normal high water, these are massive concrete or coursed-stone walls supported on spread footings or piles. The seaward face may be vertical, battered, curved, or stepped. This type is suitable for conditions of heavy exposure to wave action, and is relatively easy to construct and not too vulnerable during construction provided back filling progresses as wall construction proceeds.

(b) Slope faced. The slope faced sea walls are characterized by relatively thin reinforced concrete slabs supported on piles. The face is frequently stepped to increase wave energy dissipation and to permit

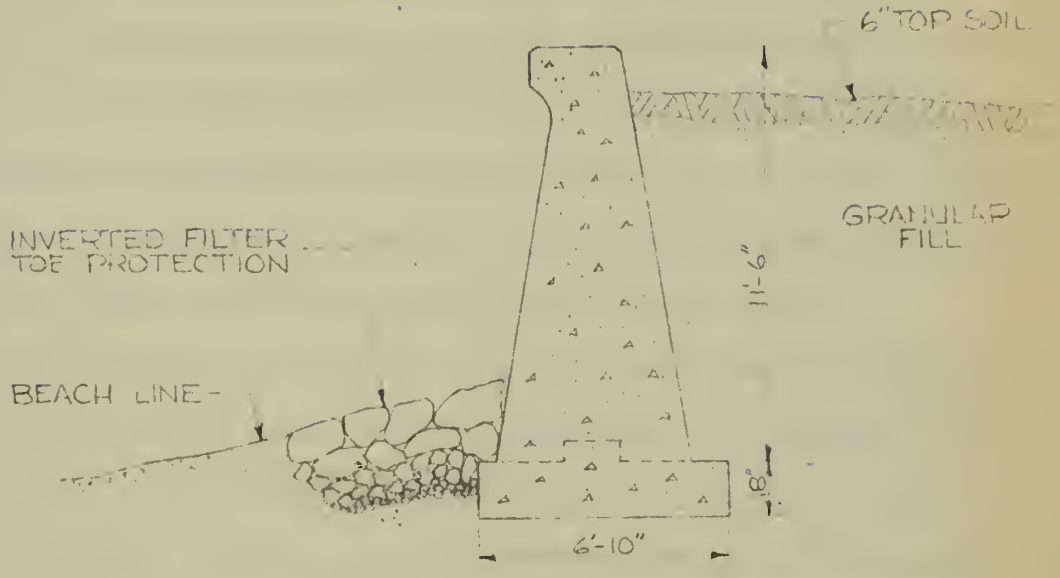


FIG. 18A
GRAVITY SEAWALL SHOREWARD OF EXTREME HIGH WATER

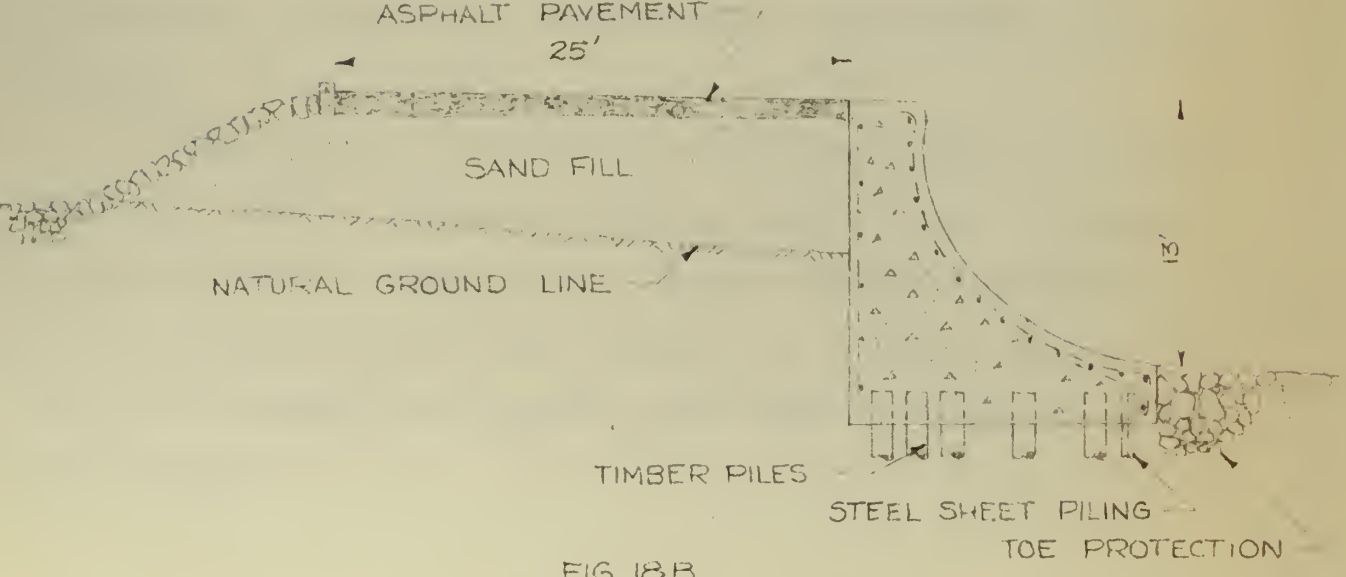


FIG. 18B
PILE SUPPORTED GRAVITY SEAWALL (Ref 51)

easy access to the beach. Walls of this type have withstood severe wave attack but are not considered as suitable for heavy exposure as type (a).

(c) Cellular sheet pile. The name adequately describes this type. Rapid corrosion of steel sheet piling in sea water generally limits the applicability of the cellular sea wall to fresh water locations. This type is adaptable to rocky bottom conditions where inadequate penetration is available to insure stability of other pile supported types.

(d) Bulkheads. Concrete, steel and timber sheet piles are used in this type of protective structure. The resistance of a bulkhead to the dynamic forces of wave action is less than that of the above types; it is therefore primarily utilized in protected locations and is particularly well adapted to wharf installations. The bulkhead is also frequently used in exposed locations above the extreme high water level to increase slope stability and prevent erosion at the toe of the slope.

(e) Rubble mound. As indicated by its name, this type is composed of random placed stones, blocks or special armor units on a core of finer material, usually constructed as an inverted filter. Wave energy is destroyed by percolation and friction within the many surface voids of the structure.

(f) Earth dike. This is one of the oldest forms of sea wall and had been utilized to protect low lying agricultural lands for centuries. Dikes are normally constructed of a compacted clay with the surface protected by a revetment of concrete slabs, stonework, or asphaltic treatment. Hydraulic fill with asphalt surfacing has recently come into accepted use within this category.

Selection of the proper sea wall type depends on the following factors:

(a) Purpose to be achieved.

(b) Difficulties and risks of constructing works exposed to tidal conditions and wave action

(c) Problems of material availability and resistance to natural destructive forces

(d) Wave exposure

(e) Foundation conditions

(f) Architectural requirements

(g) Comparison of annual costs

Consideration of the purpose to be achieved will not usually lead to selection of a specific sea wall type for the job but will frequently eliminate one or more types from further consideration. For example, an earth dike would probably not be used behind a recreational beach to protect valuable shore properties because of the width and height required. A gravity wall or bulkhead would certainly present a more logical choice for this situation. Similarly, a permeable rubble mound structure would be of little value in protecting low agricultural lands from inundation; the dike being an obvious choice for this situation.

The difficulties and risks of construction not only influence the selection of sea wall type but also must be considered in the final design. Walls in locations exposed to tidal action and wave forces must be of plain straightforward design which encourages speed in construction. This is particularly true in the lower levels. Foundation trenches are difficult

to pump dry and are subject to filling with sand or shingle by succeeding tides. This limits the depth and length of excavation to that which can be excavated, dewatered and back filled with concrete during one tidal cycle. Use of a cofferdam for developing deep foundations leads to major problems since this temporary structure may be subject to the very forces for which the sea wall has been designed. Reinforcement for concrete in lower levels must be capable of prefabrication into units which can be easily placed during low tide. The type of sea wall selected must be able to resist wave forces at high tide levels during all phases of construction. This may necessitate incorporation of protective bulkheads or cut-off walls to seal off each unit of work and to limit damage should a storm strike the area during construction. Access for construction equipment must also be considered. Frequently the sea wall provides the only safe means of access to the area. This could result in selection of a rubble mound or cofferdam type of structure.

The need for considering availability of materials is obvious. For example, possible sources of resistant rock must be considered before specifying a rubble mound structure. Equally important is the consideration of local factors affecting the life of construction materials. The possibility of marine borer infestation will eliminate the use of unprotected timber structures. Some coastal areas have proven to be especially conducive to corrosion of steel and if steel is to be used, provision for protective coatings or cathodic protection must be contemplated. A great deal has been written concerning the action of concrete in sea water. Concrete is generally a very satisfactory material for sea wall construction. With proper

care to insure dense, impermeable concrete in place, it can be expected to have a long useful life. In severe weathering conditions, air entrainment is essential and in warm waters use of a Type II cement with low tricalcium aluminate appears warranted. For concrete exposed to sea water, a minimum cover of 3" must be specified for all reinforcing bars with 4" cover at exposed corners. Recently there has been a rapid trend toward prestressed concrete for marine structures. This material has good durability in sea water, excellent structural properties and exhibits over-all economy. It has proven expecially adaptable for piling because of its strength and ability to withstand hard driving. Prestressed sheet piling will have many applications in sea wall design where appearance is a major factor.

The factors, wave exposure and foundation conditions, will be discussed in detail under the design considerations for the various specific sea wall types. Little guidance can be provided concerning architectural requirements; however, a sea wall designed to work satisfactorily with the forces of nature in accomplishing the desired function will normally turn out to be very acceptable architecturally. The reader is referred to the many books on engineering economics for a complete discussion on the comparison of alternate proposals by the method of annual costs.

C. Design Considerations

The design of a sea wall is far from being an exact science because of variation in the magnitudes of the applied forces and the many difficulties inherent in their evaluation. Studies of sea wall failures, however, emphasize the importance of the following factors in all designs:

(a) Undermining of the toe in gravity walls and rubble mound structures must be prevented. Similarly, in pile supported sea walls and

those depending on pile penetration for stability, loss of material from the seaward side can not be tolerated.

(b) Excessive hydrostatic pressures behind the sea wall must be eliminated to reduce high toe pressures in gravity types and outward pressures of high magnitude in bulkhead and slope-faced structures.

(c) Failure of the backfill by overtopping or flanking with the resultant loss in passive resistance to wave forces must be prevented.

(d) Circular slope failures of all sea wall types and differential settlements which would result in cracking of rigid walls with subsequent loss of backfill must be prevented through correct foundation design. This necessitates a thorough soil investigation of the area. This latter requirement has been too frequently overlooked in the past.

These general causes of failure have been known to the profession for many years. In view of the disasterous consequences which result from failures however, responsible engineers have tended to alter proven safe design practices very slowly with the result that full advantage is not always taken of recent advances in knowledge from model tests and prototype observations. In the discussions which follow concerning the design considerations for each type of structure, an attempt will be made to apply the data contained in Chapters I, II, and III to the elimination or reduction of those factors listed above which are the most frequent causes of sea wall failures. Many design factors are applicable to several or all of the sea wall types. These general considerations are discussed in the following sections where they seem most relevant.

TYPE I. Gravity Walls. Location with respect to extreme high water and

shape of cross section are two important considerations in this type of sea wall. The former determines the magnitude of wave forces which must be resisted. The combination of the two factors establish the height of run-up, amount of overtopping and scour potential of the structure. Using location as a parameter, two designs will be considered as follows:

(a) Structure to seaward of extreme high water

(b) Structure to shoreward of extreme high water

Gravity Sea Wall to Seaward of Extreme High Water

(1) Height of Sea Wall. One of the basic questions in all sea wall design is whether to provide sufficient height to preclude all overtopping or to permit some overtopping under extreme conditions and provide positive means of backfill protection and drainage. Economy frequently dictates the latter course; however, where the difference in total cost is not great, a design which prevents overtopping is certainly desirable. The additional cost of the higher (or more efficient) sea wall may be more than offset by the requirement for initial protection and continued maintenance of the back fill where overtopping is allowed.

The curves of Fig. 8 provide values of relative run-up for H/T^2 values of 0.05, 0.10, and 0.30, for various shapes of cross section, and for various depths at the toe. These curves were developed for sea walls fronted by a 1 on 10 beach slope. As long as the toe depth is greater than approximately 3 times the deep water wave height, this pre-slope has little effect on run-up and the curves may be used directly. For shallower toe depths which is the usual case for a sea wall, the relative run-up will increase as the pre-slope steepens and will decrease with flatter pre-slopes. While additional model tests will be required to establish the exact effect

of the change in preslope for shallow depths at the toe, the data available in the curves permits a much closer approximation of necessary sea wall height than has been available before. The range of Ho/T^2 values covers the higher and steeper waves which will be of practical interest to designers. Since the exact prediction of wave characteristics is impossible, it is considered that use of the curves is justified in obtaining a useable sea wall height. As a rule, a minimum height equal to run-up plus 2 feet should be used.

Tests by the Beach Erosion Board have shown that roughness on a gentle slope is an effective means of reducing run-up. The use of riprap on the slope in front of the sea wall is therefore recommended. Insufficient research has been accomplished to date to permit a quantitative prediction of run-up reduction through use of riprap, but in general, roughness will be most effective on the flatter slopes and for toe depths of less than $3H_o$. The roughness created by stepping the face of a gravity sea wall located seaward of the high water level is not particularly effective in reducing overtopping and incorporating a more effective roughness would result in additional costs not commensurate with the value received. Gravity sea walls are therefore normally constructed with a smooth face and sufficient height is provided to prevent overtopping from the resulting run-up.

(2) Wave Forces. Knowing the design wave for the sea wall location, the breaking depth is computed by use of the Iverson curve, Fig. 7. Because of the scattering of wave height in any wave train, it should be assumed that waves will break in a depth up to 1.5 times the value obtained from the curve. If the depth at the toe is greater than this quantity, the

force is computed using the Clapotis pressure diagram shown in Fig. 11. For lesser depths, the waves will break on or seaward of the structure. Those in the latter category will reform as smaller waves until they finally break on the structure. In this case, the maximum wave which will break on the structure as determined by

$$d_b = 1.3H_b$$

should be used for design and the breaking wave pressure diagram shown in Fig. 12 applies. For other than vertical faces, the modifications indicated on page 35 are used to determine the resultant wave forces.

In very shallow water, all but the smallest waves will break to seaward of the structure. In this case both the surge pressure as discussed on page 34 and the breaking wave pressure are calculated; the more unfavorable value being used in the design.

(3) Profile of Face. Referring again to the run-up curves, Fig. 8, it is readily apparent that the most effective sea wall shape is the curved reëntrant. Faces having a slope of sufficient steepness to be practical in a gravity structure are conducive to high run-up and should be avoided. The vertical face offers low space requirements and ease of construction at the expense of high run-up and maximum wave thrust.

(4) Foundations. As indicated previously, sea walls of this type are either founded on spread footings or on piling. In either case, erosion at the toe of the structure is a critical factor.

Where a spread footing is to be utilized, it must be located below the maximum erosion which can be predicted during the life of the structure. This erosion depends on the sea wall face, wave exposure, and prevailing

littoral conditions. Case histories of various areas show that there is no justification for assuming that a stable beach will remain in that status over a period of years. The sea wall itself will usually contribute to an immediate reduction in beach stability. For short term effects, it can be assumed that a structure located seaward of the extreme high water line will experience scour below the natural bottom to a depth equal to the height of the maximum unbroken wave that can be supported by the depth of water at the toe of the structure. The standing wave produces a rapid alternation in hydrostatic pressures at the beach level and thus causes extreme turbulence with the resultant tendency for rapid scour. In shallower water where most waves break on or seaward of the structure, the short term effects can be approximated as producing scour to a depth below high water line equal to the height of run-up on the structure. This result is caused by the impermeable sea wall's preventing percolation of the wave surge into the beach as would normally occur without the sea wall, and the increased velocity of the return surge resulting from the relatively higher run-up on the sea wall as compared to a normal beach slope. An additional factor occurring where the back fill becomes saturated through overtopping or collection of run-orr is the upward seepage force which takes place at the toe. The combination of large quantity and high velocity of back wash with upward seepage forces promotes rapid scour. As scour progresses, the height of wave reaching the wall increases and the tendency for scour is augmented. The material near the wall will be dragged seaward flattening the beach slope and extending the step. Where the wall is above normal high water between storms, this material will be alternately built up and removed. For walls in deeper water, the original material will not return

and the beach will gradually become submerged. Model studies have indicated that depletion of a beach by wave action ceases at a level of about one wave height below low water.

Loss of material at the toe of the sea wall can be reduced by utilizing an inverted filter as shown in Fig. 18. This has the double advantage of minimizing seepage effects and reducing run-up with the resulting reduction in return flow velocity. Use of large stones without a filter base will not be effective since the finer materials will continue to scour allowing the stones to sink. Use of an apron (a remedy frequently employed in the past) should be avoided since run-up is increased and permeability reduced. A sheet pile cut-off is recommended at the toe of the spread footing to provide additional insurance against undermining and to lengthen the seepage lines thus reducing the tendency for scour. The lower portion of a construction cofferdam when utilized is often left in place for this purpose. Figure 19 shows an example of staged construction in which the original reinforced concrete sheet piles will be cut off and will serve as toe protection for the final gravity structure. This figure also shows an interesting means of handling a severe drainage problem.

The above discussion applies in general to sea walls constructed above normal high water level and fronted by a relatively stable beach. If the foreseeable littoral regimen indicates that erosion of the beach will occur, it is necessary to estimate the amount of erosion which will take place during the life of the structure. Records may indicate the average loss of beach width per year or littoral studies can be used to predict the loss of material from the area in cubic yards per year. This latter quantity is converted to loss in beach width by assuming that "loss

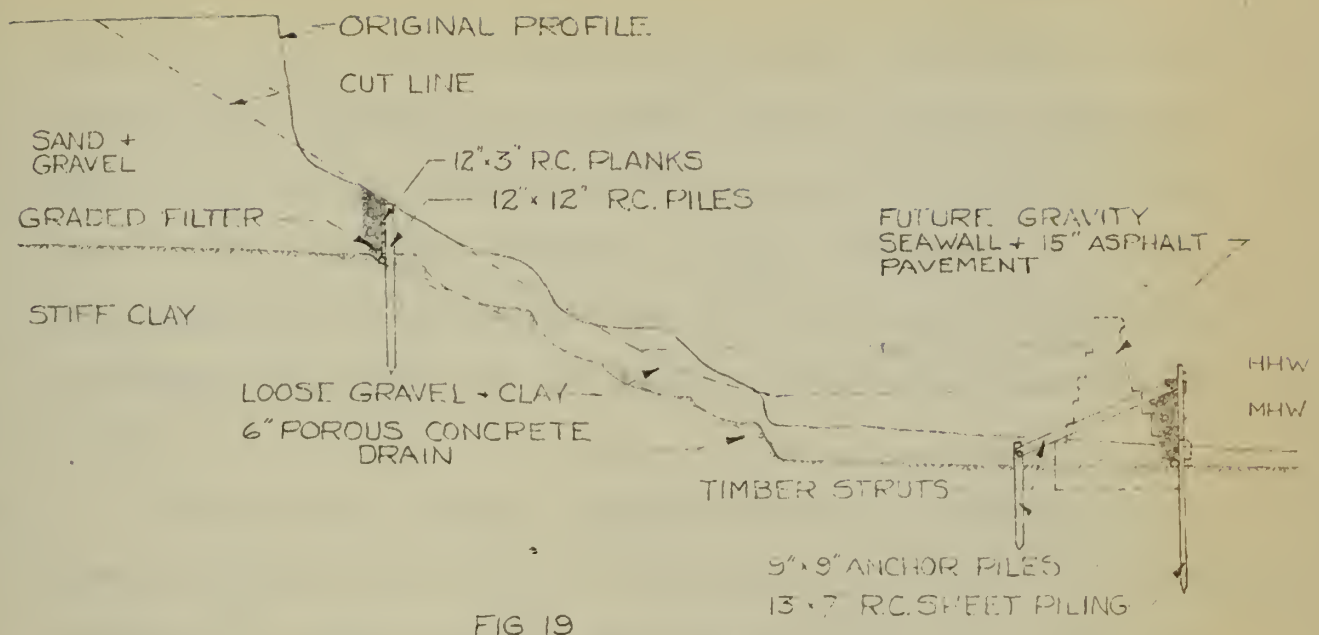


FIG 19
STAGED CONSTRUCTION AT CLACTON, ENGLAND (Ref 39)

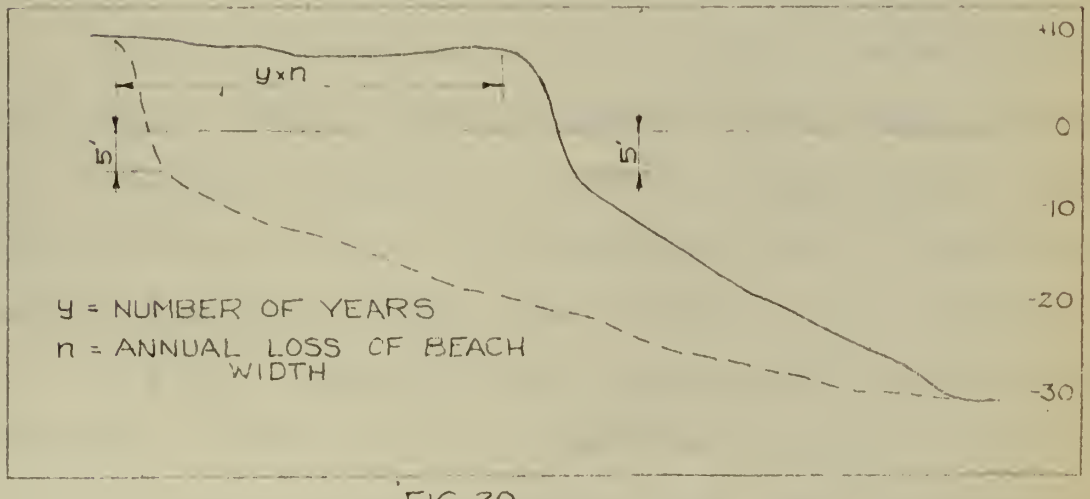


FIG 20
BEACH EROSION PREDICTION (Ref 4)

of one cubic yard of beach material equals loss of one square foot of beach area at the berm." The shoreward recession of the beach profile can then be predicted by assuming that the profile above the break in slope which occurs below the step (approximately 5 feet below MLW) remains unchanged and that this slope break remains at a constant elevation. See Fig. 20. This same method can be applied to the short term effects of storms on a relatively stable beach where the reduction in berm width is a matter of record.

The sea wall located to seaward of normal high water presents special problems since scoured material will be carried away by the littoral currents. To prevent this loss, groins may be required not to build up the beach, since a sea wall in this undesirable position would not be required if sufficient littoral drift were available, but to maintain the existing material in position by preventing its lateral displacement. The use of groins for this purpose will be treated separately in a later section.

With the necessary depth of spread footing to prevent undermining, the foundation design is similar to that for a normal gravity retaining wall considering, in addition, wave forces and excess hydrostatic pressures which may occur. In view of the probability of scour, no passive pressures should be considered as acting on the seaward face of the structure.

The above discussion on scour applies equally to pile supported structures. In this case, however, sufficient penetration to resist the overturning moment becomes the critical factor. Use of an inverted filter in front of this type of wall is essential. A sheet pile cut-off must always be utilized since even small amounts of scour will allow escape

of the backfill. To insure that loss does not occur through joints in the sheet piling, the following rules are recommended:

(a) With reinforced concrete sheet piles, the tongue and groove must extend to the top. Leaving a double groove to be grouted after driving has proven unsuccessful in many cases.

(b) An inverted filter should be placed behind the sheet piling near the top to prevent loss of fine materials.

In the general area of foundations for gravity structures, all of the factors for retaining wall foundations must be considered in addition to the special factors indicated above. The worst possible combination of earth forces, hydrostatic excess pressures and hydrostatic uplift, must be applied in investigating wall overturning seaward. This would normally occur with no water against the face of the wall and with a saturated backfill. For stability against overturning landward, maximum wave action and unsaturated backfill condition should be checked. Hydrostatic uplift is usually computed on a straight line distribution based on assumed still water level at the toe and ground water level at the heel. Careful soil exploration to insure against excessive differential settlement is essential and where slopes are to be stabilized, investigations of stability against circular shear failure must be accomplished.

(5) Excess Hydrostatic Pressures. When the fill behind a sea wall becomes saturated through overtopping or through collection of ground water, each wave trough presents a rapid draw down condition with the water pressure shoreward of the wall unopposed to seaward. This causes high toe pressures which may result in failure. These excess hydrostatic

pressures must be prevented by adequate drainage and by excluding the possibility of overtopping water's entering the backfill. Transverse drains should not be allowed above low water level since they would be subject to high pneumatic pressures under the action of breaking waves. Where drains are used, filters must be provided to prevent loss of backfill. Likewise, all expansion and construction joints must have positive means for prevention of sea water ingress and backfill loss. Filters behind the sea wall in these areas are desirable.

(6) Backfill Erosion. The best method of preventing this common cause of failure is to provide a sea wall of adequate height with a reentrant face. Wind blown spray will still be a problem; however, erosion from this source can be prevented by protecting the backfill with a watertight pavement, carried back at least 25 feet or to the point at which the berm begins to slope downward. The back slope should be provided with a natural cover and positive means of erosion prevention at the toe.

Where overtopping occurs, pavement will not provide adequate protection because of hydrostatic uplift pressures. In this case, the backfill should be constructed as an inverted filter with stones of sufficient size at the surface to resist uplift. The modified Iribarren formula on page 36 applies. Drainage requirements for this case are excessive.

To prevent flanking, vertical bulkheads should be used at the ends of a sea wall. These bulkheads must extend shoreward beyond the maximum recession anticipated along the unprotected shoreline. Where sea walls of great length are to be constructed, intermediate bulkheads extending shoreward at intervals will localize any failure which may occur. This addi-

tional insurance should not be necessary after the completion of construction if proper attention is given to the original sea wall design.

The problem of eroding backfill behind a gravity structure is so frequently encountered that the question arises of whether it should be considered in design for stability against wave forces. It is considered that the passive pressure of the backfill should be utilized in the design provided that positive means for preventing erosion are incorporated in the finished work.

Gravity Sea Wall to Shoreward of Extreme High Water

(1) Height of Structure. With a sea wall in this location, much of the wave energy which produces run-up will have been dissipated on the slope in front of the structure. The $d/H_o = 0$ curve of Fig. 8 applies for the structure toe at the high water line. As the location of the structure moves shoreward, the run-up on the structure decreases until all run-up is dissipated on the pre-slope at a point where the toe of the sea wall is located at the maximum height of run-up on the slope alone. Between these two extremes, the following equation can be used to determine the height of run-up on the structure:

$$R_s = R_1 - (R_1 - R_o)\frac{d}{R_o} - d$$

where R_s = run-up on the structure

R_1 = run-up on the structure where depth at toe = 0

R_o = run-up on the slope with no sea wall

d = depth at toe, $0 < d < R_o$

As with the sea wall located to seaward of the extreme high water level,

the height of sea wall should be equal to run-up, R_s, plus 2 feet.

(2) Wave Forces. Surge force of the broken wave which applies in this case is computed as outlined on page 34.

(3) Profile of Face. Overtopping is not as great a problem with a sea wall in this location and can be avoided by a relatively small addition in height when using a vertical face in lieu of the curved re-entrant face as recommended for the gravity wall seaward of extreme high water. In view of its ease of construction and small space requirement, the vertical faced wall is recommended for this case.

(4) Foundations. A wall located between normal high water and the crest of a stable beach will result in scour and general flattening of the beach as previously discussed. When located above the beach crest, a sea wall causes increased scour only during storms and the beach can be expected to build up to its previous level during fair weather. There is no general rule for predicting the amount of scour which will occur in either case and the important thing is to prevent any recession of beach level at the toe.

Model tests have shown that roughness has a greater affect in reducing run-up on gradual slopes and in areas where the surge depth is small. This indicates the advantages of utilizing a rubble surface in front of the gravity wall. Stone size as before is determined from the modified Iribarren formula given on page 36. Even with this precaution, the footings or pile tops should be well below the beach level.

The discussion of sea wall foundations on an eroding beach contained on page 67 applies here. It is axiomatic that a sea wall must be designed not only to meet present conditions but also for conditions which

will exist during its useful life.

(5) Excess Hydrostatic Pressures. The earlier discussion applies. Excess hydrostatic pressures must be eliminated by preventing overtopping, providing adequate back fill drainage and barring the access of water to the back fill from the surface or through joints in the sea wall.

(6) Back Fill Erosion. Where adequate height of sea wall is provided, natural protective cover should provide sufficient protection against erosion. In this connection, it is worthy of note that use of a bitumen emulsion will produce a high quality grass surface in a short period. The seed bed is prepared by rolling and moistening if necessary. Approximately 1" of clean sand is then spread and a rapid breaking emulsion is sprayed on. The film retains moisture in the surface soil and maintains higher than normal temperatures thus promoting early germination and root formation. In addition, the film prevents seed loss and erosion during the initial period. A bitumen emulsion application rate of 1/6 gal. per square yard should give satisfactory results.

TYPE II. Slope Faced Sea Walls

Structures of this type as shown in Fig. 21 are normally pile supported and require much less material than a gravity structure. A slope faced sea wall is usually built with the upper surface of the toe at extreme high water level. The grade beam is carried several feet below beach level at this point. When uniform slopes are used in this construction, the run-up formula on page 28 applies. Stepping the face will reduce run-up and toe erosion to some extent. Access to the beach will be facilitated by a stepped face and where this is a factor, the resulting additional cost may be war-

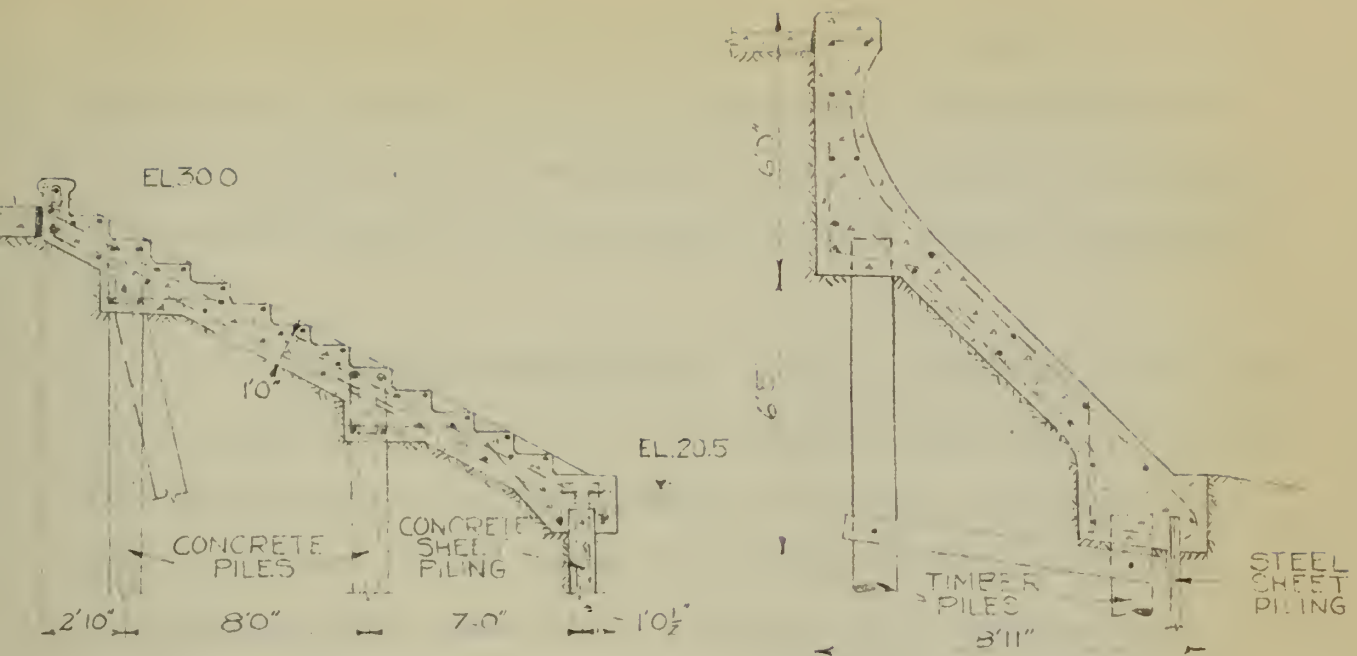


FIG. 21
SLOPE FACED SEAWALLS (Ref 51)

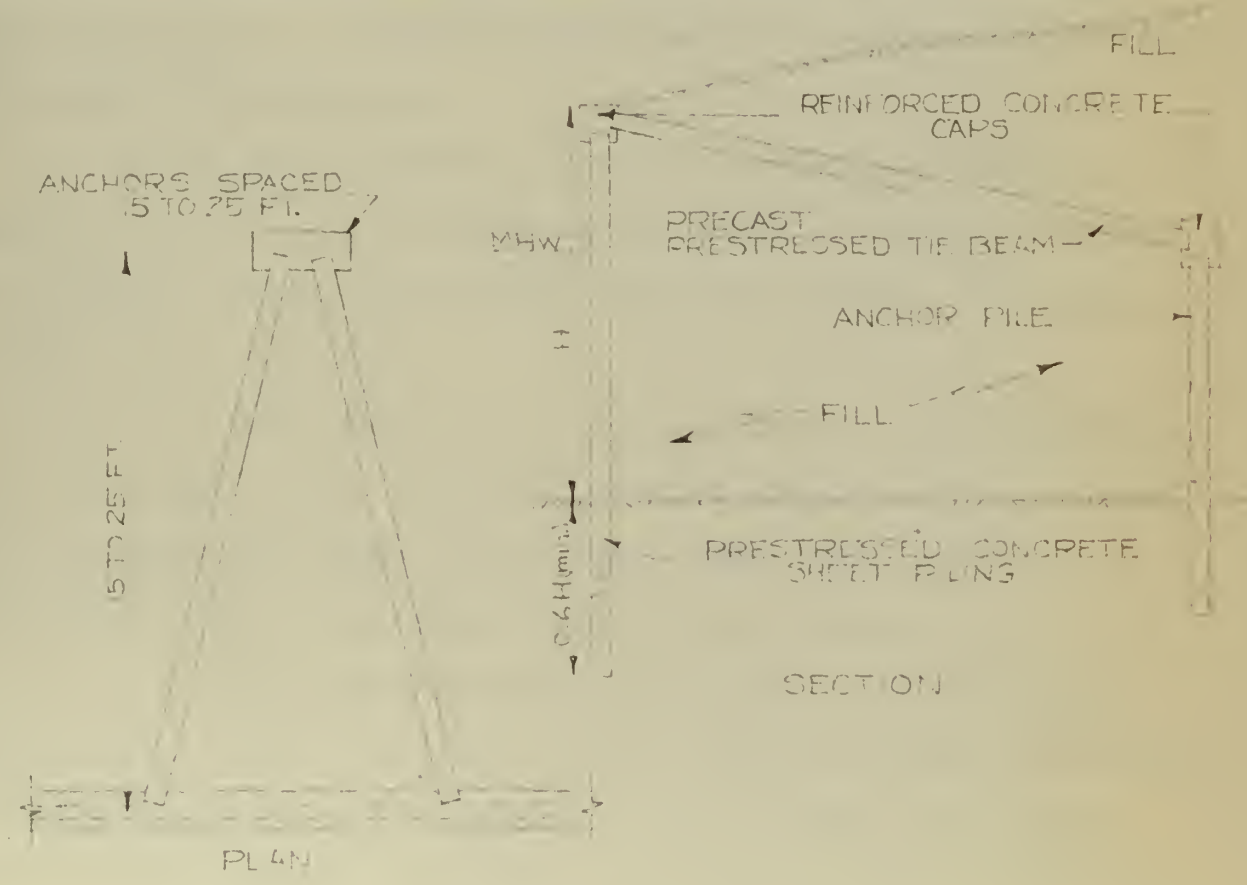


FIG. 22
PRESTRESSED CONCRETE SHEET PILE BULKHEAD (Ref 44)

ranted. For other than uniform slopes, the method of computing run-up as developed for Type I(b) sea walls is recommended. The requirements for height, toe protection and prevention of back fill erosion are the same as in the gravity sea walls. Wave pressures are also computed as previously indicated.

(1) Structural Considerations. The basic difference between slope faced and gravity sea walls lies in the structural design since the sloped face sea wall does not depend on weight for stability. Failures of this type of structure are normally characterized by seaward movement of the slab which provides a basis for the following design recommendations:

(a) Although the structure has been made sand tight and scour at the toe prevented, the fill will always settle to some extent pulling away from the face. It is therefore not possible to depend on the back fill for support of the face slab.

(b) Excess hydrostatic pressure is extremely difficult to prevent with this type of structure and the face will therefore be subject to alternating wave forces and hydrostatic uplift pressures. This stress reversal must be considered.

(c) The junctions of the face slab with the supporting piling will act as rigid joints. The entire structure must be treated as a rigid frame to insure that cracks will not form at these points exposing the reinforcing steel to corrosion and inviting eventual failure.

(d) Even with the most careful precautions. loss of backfill may occur as a result of an extreme storm. To preclude failure in this case, the supporting piling must be designed to act in bending. The point of fixity for the supporting piling under each loading condition is taken at

the assumed surface of the backfill.

TYPE III. Cellular Steel Sheet Pile Sea Walls

As indicated previously this type of sea wall has limited application in sea water but is valuable in relatively protected locations where insufficient depth to rock prevents use of a normal bulkhead. Design of the cellular structure is carried out as described in Ref.42 with the additional factor of dynamic wave forces computed as described under Type I sea walls.

It is reasonable to assume saturation of the fill material to the maximum height of run-up on the seaward side and to the natural ground water level to shoreward. Keeping the natural ground water level low is important to stability of the structure and therefore the previous discussions on protection of backfill apply equally to the cellular sheet-steel sea wall.

(1) Overtopping. Loss of fill material from the cells of this type of sea wall is probably the most important item to be considered in design. The shape of the face results in a concentration of wave energy at points where the connecting arcs meet the cells. Although model tests have not been run to determine the increase in run-up on a structure of this type, it is anticipated that at least twenty per cent higher run-up will be experienced than with a vertical faced structure. Where diaphram type cells present a fairly uniform face, this additional run-up will be much less and the factor corresponding to a vertical face can be used safely.

Prevention of overtopping by providing adequate height is the best method of preventing loss of fill material. It is very risky to assume that no higher waves will ever occur than those which we can predict at present. It is therefore recommended that one of the following methods be

adopted to provide additional protection:

(a) Provide a reinforced concrete cover-slab. This slab should be supported on the sheet piling by welding the reinforcing steel to the piling or by some other means. This will prevent settlement and possible cracking of the slab. An access hatch is desirable for replenishment of the fill, should settlement become excessive.

(b) Utilize cap stones of sufficient size to prevent displacement, should the structure be overtopped. One method of determining this size is to assume that the velocity of the overtopping water will equal the velocity of the wave at the breaking point. The Isbash Formula,

$$d = 0.08V^2$$

can then be applied. This formula is based on a specific weight of rock equal to 165 lb. per cu.ft. and will yield a stone of sufficient size to resist overturning. (Ref. 56) A much larger size would be required to resist sliding; however, it is assumed that confinement within the cell and careful bedding to promote interlock will provide adequate safety against this factor.

TYPE IV. Bulkheads.

Like the cellular sheet steel sea wall, the sheet steel bulkhead is subject to rapid corrosion in a salt atmosphere and therefore finds greatest application in fresh water locations. Timber bulkheads are subject to the same restriction not from corrosion but from their susceptibility to marine borer attack. Timber can be used successfully above the high water line where subsequent erosion cannot expose it to future borer attack. Concrete sheet piling is applicable to both salt and fresh water locations; however,

abrasion must be considered where a shingle beach fronts the structure.

As a general rule, bulkheads serve the primary purpose of retaining a fill and when designed for this purpose will be safe against dynamic wave forces. Bulkhead design methods for penetration in both granular materials (fixed earth support) and clays (free-earth support) are contained in reference 55. Computation of wave forces is handled as described under sea walls Type I(a). Run-up from Fig. 8 for a vertical wall applies to bulkheads located below the high water line. Shoreward of this point, the method developed under sea wall Type I(b) may be utilized. Calculated height of run-up plus two feet is usually used as a minimum height for this type of structure. Loss of fill material either through the sheet piling or by erosion as a result of overtopping or flanking will destroy the bulkhead's stability against wave forces and result in its ultimate destruction. To provide insurance against this tendency, tie backs and anchor piling should be capable of working in both tension and compression and must be located in such a position that their ability to function can never be impaired by erosion of the backfill. A recent bulkhead job which illustrates this principle is shown in Fig. 22.

(1) Toe Erosion. Stability of a bulkhead depends on passive earth pressures seaward of the toe. For this reason, adequate penetration must be provided to insure development of these passive pressures under the most unfavorable conditions of erosion. In addition, positive means of preventing erosion in the form of an inverted filter of riprap at the toe must be employed.

(2) Backfill Erosion. The prevention of backfill erosion is of crucial importance to the stability of this type of structure. Where any

possibility of overtopping exists, paving must be utilized. Adequate drainage of surface run-off is essential. To prevent loss of backfill through the sheet piling, filter material should be placed behind the bulkhead. Grouting of concrete sheet pile joints is not recommended; instead, the tongue and groove detail should be continued to the top.

(3) Excess Hydrostatic Pressures. Prevention of overtopping and protection of the backfill from erosion as indicated above will be helpful in eliminating the problem of high pressures on the sheet piling from water saturation of the backfill. In addition, drainage of the backfill should be provided where possible and granular material must always be placed immediately behind the sheet piling. In Tschebotarioff's tests at Princeton, it was shown that a sand dike on a natural slope behind the sheet piling was very effective in reducing lateral pressures when the backfill was to be a fluid clay. (Ref. 54) This type of construction is recommended for all bulkhead sea walls. The bulkhead design shown in Fig. 19 meets the general requirements outlined above.

TYPE V. Rubble Mound Sea Walls

Like all sea walls, protective structures of this type must satisfy the following design requirements:

(a) They must be stable against slope failure.

(b) They must be capable of resisting wave attack.

(c) Wash out of the backfill must be prevented.

(d) The structure must not contribute to the erosion of the slope in front of the structure.

(e) Materials used in construction must be able to withstand the forces to which they are exposed.

(1) Stability. The rubble mound is basically a flexible structure as compared to either the gravity or slope faced types. Its ability to settle without failure renders the rubble mound particularly adaptable to poor foundation conditions. Unfortunately, placing a blanket of heavy stones on a slope of poor materials greatly increases the possibility of shear failure. Stability against a circular earth failure must be carefully checked and measures taken to prevent such an occurrence. These measures must include provision for drainage above the sea wall and positive erosion control at the toe.

(2) Drainage. The rubble sea wall offers no obstruction to the flow of water from the backfill. To prevent loss of fine materials through the structure and to prevent concentration of flow lines at the toe of the slope, filters are required. Where a clay slope is being protected, "herringbone drainage" can be used to advantage. This consists of breaking up the slope through use of gently sloping berms. A French drain is placed along the inner edge of each berm. Transverse drains intercept these at intervals to conduct the run-off to the base of the rubble sea wall. The slope between berms should not exceed 1 on 2. A natural protective cover on the slope and well back from the crest is essential. Use of a bituminous emulsion as previously described is recommended. A clay slope underlain with pervious layers may require the installation of horizontal drains to prevent uplift and subsequent slope failure. Sources of artificial seepage must be removed or adequate drainage installed to intercept the slope-weakening flow. The key to slope stability is good drainage which can only be achieved through an adequate soil exploration program.

(3) Toe Erosion. Toe erosion reduces the resisting moment of the

slip circle used in checking the over-all slope stability and causes failure of the rubble mound as toe stones are undermined and roll forward into the void. These serious effects must be prevented.

The discussion of erosion prediction under gravity sea walls is especially applicable in considering use of a rubble mound structure. As a general rule, a sea wall of this type must never be located where undermining of the toe can occur.

The rubble mound sea wall is basically a non-erosion producing structure. The rough and porous surface causes maximum dissipation of wave energy thereby reducing erosion in front of the structure to a minimum. To insure this effect and to improve the over-all slope stability, the toe should be carried well forward below the beach line as shown in Fig. 23. The stones at the toe should be at a depth below storm water level at least equal to the height of the maximum wave which will break on the structure and for structures in shallow water, never less than the height of the run-up.

(4) Height and Cross-sectional Shape. A rubble mound sea wall must have sufficient height to prevent overtopping. To achieve this result, the method of run-up prediction developed on page 28 is used. It is obvious that the run-up, and therefore the required crest height, depends on the shape of the cross section and the location of the structure on the shore profile. Normally a rubble structure will not be utilized above the extreme high water line since previously discussed types of sea wall are more economical and equally effective in this location. In shallow water, a simple slope will prove most economical since the wave energy will have been par-

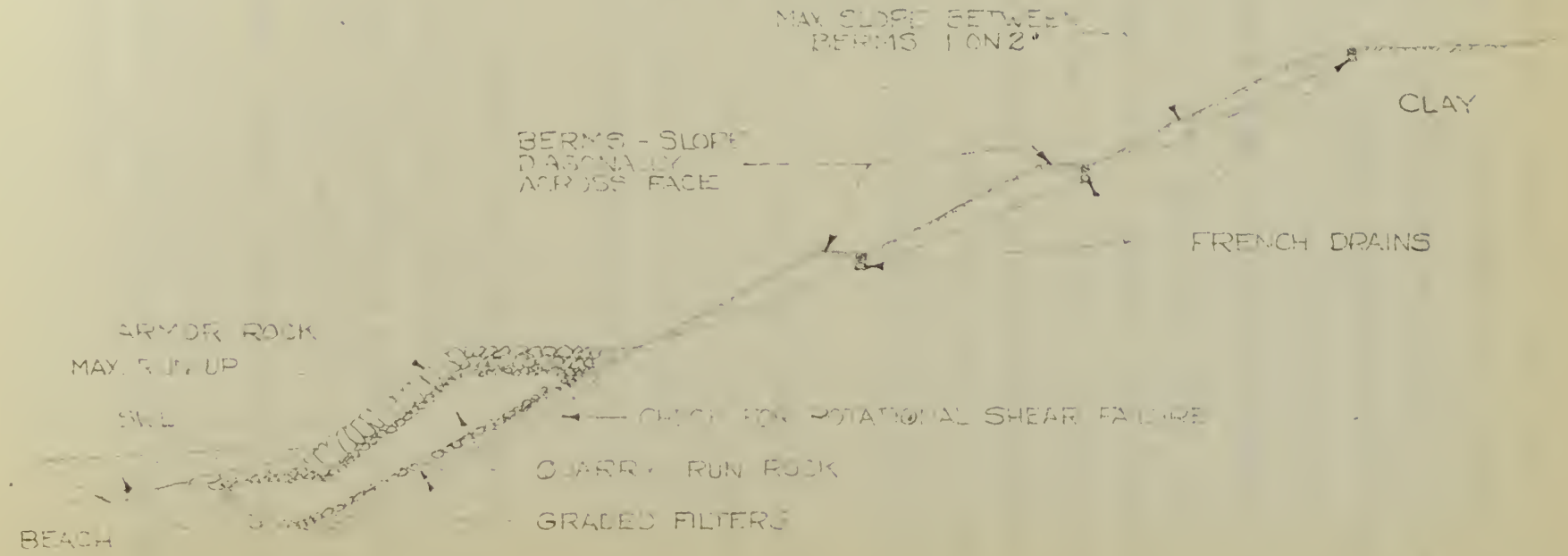


FIG. 23

PROTECTION OF CLAY SLOPES WITH PEBBLE MOUND AND HERRINGBONE DRAINAGE (After Ref. 36)

tially dissipated on the natural shore slope. The simple slope has the additional advantage of requiring the least base width as compared to composite or berm profiles. As the depth in which the structure must be placed is increased, the desirability of providing a composite slope or berm becomes pronounced since the simple slope experiences much greater run-up and therefore must reach a greater height to prevent overtopping. The berm type profile requires slightly more base width than a simple slope but results in reduced cross-sectional area. A composite slope requires the greatest base width, gives the lowest crest elevation and results in either the greatest or the least cross-sectional area depending on whether a steep or shallow slope is used to seaward. Figure 9 illustrates the savings in material resulting from utilizing a flat slope below storm water level followed by a steep slope above.

In any case, the initial slope of the structure must be less than $(H/T^2)^{1/2}$ to insure breaking of the impinging waves. When using either a composite or berm profile, the break in slope or berm height should be located at the storm water level to give maximum energy dissipation at minimum construction cost.

(5) Wave Forces. In a sea wall of this type, resistance to wave attack is achieved by providing armor units of sufficient weight to withstand uplift. Usually, the armor units employed in rubble mound sea walls are selected quarry stone having a high specific gravity (2.65) and high resistance to abrasion and seawater attack. The size of the waves which will finally reach a sea wall does not ordinarily require the use of heavier artificial units such as tetrapods or tribars, both of which have proven very successful in breakwater construction.

Where the sea wall is located in fairly deep water so that the H/d value is small, the new formulae developed by the Waterways Experimental Station for unit weight, layer thickness and number of units should be used. See page 38. In shallow water, the modified Iribarren formula can be used to compute unit weights until additional information is published which will relate the new formula to this situation. Formulae for layer thickness and number of units as indicated above will be of practical use in this latter situation.

TYPE VI. Dikes.

The outstanding feature of dike construction is impermeability. This is achieved by constructing the core of impermeable clays or by utilizing an impermeable surface layer on a core of sandy materials. A dike is extremely vulnerable to wave attack and therefore a protective revetment is required. This revetment may consist of natural stone or cast armor units, concrete slabs or asphalt treatment. Sea walls of this type have frequently failed after overtopping which resulted in back slope erosion. Hence, the general rule: the back slope of a dike must be given as careful consideration as the face exposed to the sea.

(1) Structural Profile. The profile of the dike determines the required strength of the revetment and the height to which the structure must be carried. Hunt's work (Ref. 13) provides the best approach to the problem of profile selection. His basic formulae as previously noted were developed for smooth impermeable slopes. While the normal dike surface is generally made impervious as discussed below, it is seldom truly smooth. a factor must therefore be applied to Hunt's equations to account for

roughness. Roughness factors have been developed only for a 1 on 10 slope with various sand and gravel coverings. Until additional data are published on this subject, it is recommended that the following roughness factors be utilized with Hunt's formula:

0.8 - for concrete or asphalt grouted neatly-set stone or precast units

1.0 - for smooth concrete and asphalt treatments

Utilizing Hunt's method, the most economical cross section which will satisfy the site requirements can be selected. A comparison of several profiles is shown in Fig. 9, which indicates numerically the advantage of utilizing a composite slope or berm. As previously noted, the point of change in slope is made to coincide with the highest anticipated water level. Figure 24 illustrates that the principles confirmed by the Beach Erosion Board and Waterways Experiment Station tests had been previously incorporated in successful sea walls by the Dutch. An additional innovation developed in the Netherlands for reducing run-up is to build the berm in the form of a shallow trench. During a storm, water accumulates in this trench and exhibits a marked frictional resistance.

From a practical standpoint, slopes steeper than 1 on 4 are not readily accessible to wheeled vehicles and therefore repairs will be made more difficult. Berms (unless built as a trench) should have a slope of at least 1 on 20 to insure drainage. Slopes which are to rely on natural cover for protection should not exceed 1 on 2 1/2.

(2) Protective Revetments. A revetment is no stronger than its weakest point. The critical point, as in all sea walls, is at the toe. Protection of this vulnerable area is provided by fascine mattresses loaded

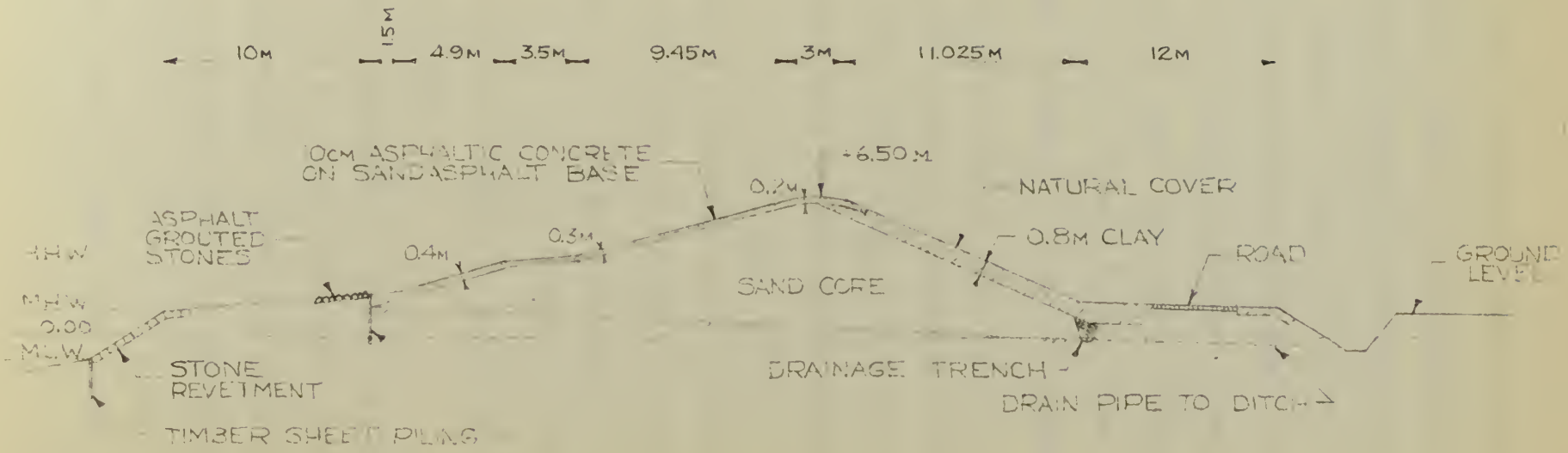


FIG 2-4

TYPICAL DUTCH DIKE OF RECENT CONSTRUCTION (After Ref 58)

with stones or inverted filters topped with rubble. Stone size can be determined from the modified Iribarren formula. Some experimental work has been done in the Netherlands with asphalt mattresses reinforced with fiber nets, woven wire or steel cables. While no definite conclusions have been reached concerning their value, it is known that trouble can be anticipated where adjacent mattresses adjoin. The toe of the revetment meets the outboard rubble normally at mean low water level. Short sheet piling is recommended as additional insurance against erosion at this point.

From mean low water to mean high water, the dike is continually subject to moderate wave attack and heavy erosion. Heaviest wave attack during storms occurs at the storm high water level resulting in high impact and uplift forces. There is no definite destinction between these two areas for purposes of design and the entire revetment from MLW to HHW should be built to resist heavy erosion and the impact and uplift forces of the design wave. When a berm is placed at HHW as recommended in the previous section, the heaviest revetment section is terminated just beyond the crest of the initial slope. The toe of the slope following the berm requires the same protection as the initial slope but for only a short distance. For uniform or composite slopes, the maximum strength revetment is carried to at least one-half the design wave height above HHW.

Waves breaking on the slope force water into all voids under high pressure. When the wave recedes, this high pressure tends to lift the revetment surface. Friction usually prevents stones from falling back into place and therefore the surface fails progressively. To prevent destruction of a dike revetment from this hydrostatic uplift, all voids near the surface must be sealed. When using natural stone or precast armor units, grouting

with asphalt or concrete is essential. Concrete grouting results in a loss in flexibility and should only be used when settlement of the dike is virtually complete. Failure to observe this precaution will result in the surface layer's bridging a settled area. Destruction of this spot during the first severe storm is a certainty. Asphalt grouting does not have this liability. It is impermeable and has good resistance to wave attack while retaining the ability to permit settlement of the surface. To avoid uplift, grout should penetrate at least 3 to 4 feet. The quantity required can be reduced by filling the surface voids with smaller stones but not to the extent that adequate penetration is prevented. Asphalt revetment grouting can be accomplished either above or below water utilizing a mix of approximately 72% fine sand, 10% filler of asbestos fiber and 18% bitumen by weight. Engineers planning to utilize this method should obtain detailed information from one of the various firms specializing in asphalt applications.

Stone size for revetments varies from those which can be placed by hand to precast blocks 1 x 1 x 1/2 meter in size. Square blocks have proved best in service. The skill required and hence the cost of placing natural stone has almost eliminated this form of protection. Precast concrete units are now being used although concrete gradually disintegrates in seawater particularly in areas experiencing freezing and thawing. Low resistance to sand abrasion and smoothness which promotes run-up are additional liabilities. Low cost, availability, and good setting quality which reduces the possibility of uplift more than offset the disadvantages of using precast concrete units.

Hot mix asphalt surfacing has been utilized extensively by the Dutch

as a protective revetment for the critical face area above low water. A dense mix of either asphaltic concrete or sheet asphalt is laid hot and consolidated by rolling or tamping. Fine aggregates are essential for the required impermeability. The surface is generally put down in two courses with the top course of 4" thickness. The base course is varied in thickness from a maximum of 12" at the toe for a sea wall experiencing heavy wave attack to 4" at the crest of the sea wall. This 4" base course is frequently continued over the crest and down the back slope. A tack coat is used between the two courses in the usual manner.

Above the termination of the heavy revetment, it is necessary only to protect the dike from attack by the uprushing storm waves. In this area both neatly set stones or blocks and asphalt surfacing are utilized. The former has the advantage of roughness in an area where water films are relatively thin and will therefore be effective in reducing run-up. Since these units must present an impermeable and flexible surface, asphalt jointing is required. For speed and economy of construction, asphalt surfacing is preferred for this area. Cast-in-place concrete slabs are occasionally used above HHW but are not recommended because of their lack of flexibility which results in joint failures and subsequent hydrostatic uplift. These unreinforced cast-in-place revetments should never be used below the storm water level or on unconsolidated dikes.

It is not safe to assume that a combination of wind and waves will not occur producing storm water levels higher than any previously recorded. For this reason, back slopes of dikes must be adequately protected against erosion as a result of overtopping. In areas of severe exposure, the 4" asphalt base course should be carried down the back slopes and extended 3'

beyond the inshore toe of the dike. Where, as a result of more favorable exposure conditions, the possibility of overtopping is remote, natural protective cover is used. The use of a bituminous emulsion as described under Type I(b) sea walls will provide initial protection of slopes to be treated in this manner and will promote a strong stand of grass. Since these dikes usually protect lowlands, which may be flooded, some form of positive protection at the inner toe must be provided. This can consist of asphalt surfacing or merely rubble on an inverted filter. Three feet both ways from the toe should be sufficient.

To insure good penetration, rapid set to avoid destruction by waves and tide, and high durability after set, hot mix asphalts have been used exclusively in revetment construction. Because of the inaccessibility of dike locations, the equipment and plant problems for this type of construction are very difficult. In addition, obtaining a good bond between hot mix and cold wet stones has not been completely successful. To overcome these disadvantages, an electrolytic process called "eshalite D.U." has been developed in Holland to produce a cold mix with all of the desirable qualities of hot asphalts. (Ref. 52) This material can be prepared with equipment which most road contractors have available and is said to have good bonding qualities. The material has not been proved in service as yet but indicates the research which is going on in this field.

(3) Core Construction. As in all sea walls, construction of an earth dike requires a thorough knowledge of subsurface soil conditions necessitating an adequate boring program. This statement has particular meaning in the present case since dikes are frequently constructed on the

poorest types of soil. Consolidation of underlying soil layers under the weight of the dike must be computed to insure that adequate initial height is allowed to compensate for the inevitable settlements. For non-flexible revetments, not only the amount but also the rate of settlement must be predicted so that revetment construction can be delayed until most of the settlement has occurred. To avoid circular failures and the formation of mud waves, the shear strength of the soil must be investigated along possible failure surfaces. At LaGuardia Airport in New York, sand drains were utilized under a dike to permit more rapid core construction than would have been possible with normal consolidation of the underlying compressible layer.

Above low water level, compaction of the clay core is an important factor in successful dike construction. Clay containing sand or fine granular materials is recommended to limit shrinkage and crack formation. The core should be built up in layers and compacted at optimum moisture content. Where a sand core is to be used, the graded surface of this material must be protected with an asphalt leveling course before the surface courses can be applied.

D. Composite Sea Walls

The preceding pages have been devoted to discussions of the various design considerations involved in each of the "pure" types of sea walls. It should be borne in mind that conditions are frequently encountered which can best be satisfied not by use of one of the basic types but rather by a composite construction embodying the most desirable features of several. The number of such possible combinations is limitless; however, the sea wall shown in Fig. 25 is a good illustration of the principle involved. This sea

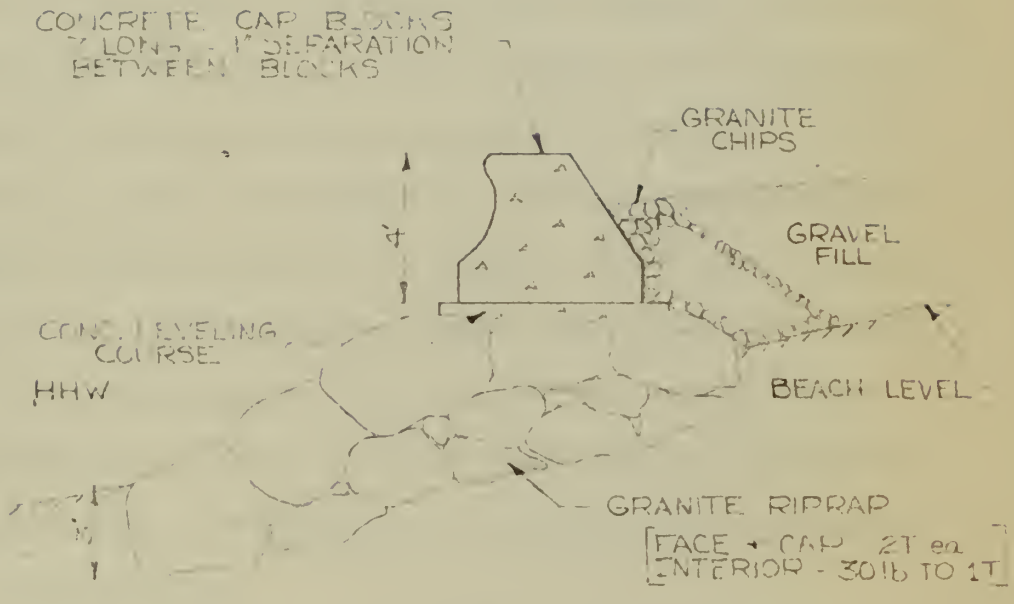


FIG 25
COMPOSITE SEAWALL - BOSTON, MASS.
(Ref 5)

wall can be constructed without use of a cofferdam, provides good backfill drainage and facilitates the production of good quality concrete in place. The design considerations listed previously for each of the individual types must be applied to a composite structure.

E. Groins

The importance of toe protection has been emphasized throughout this chapter and for that reason a short discussion of groins is imperative. Since groin application and design is a field in itself, the following comments will be limited to basic principles as applied to the use of groins in conjunction with sea walls. The purpose of a groin system is to modify the natural material-energy balance of the shoreline resulting in accretion or decreased erosion. It is important to realize that without an adequate supply of littoral materials, groins will be unable to cause accretion. While they will function to hold the existing beach material in position, this will normally be done at the expense of a reorientation of the beach profile between adjacent groins.

When use of a groin system is being considered, the following questions must be answered:

(a) Will groins accomplish the desired purpose? The answer to this question must be based on a study of the littoral processes in the area as described in Chapter III.

(b) Will the cost of accomplishing the desired purpose be less than for other equally effective methods such as artificial nourishment of the beach, sand by-passing at an updrift estuary or construction of an off-shore breakwater?

(c) What will the effect of the groins be on the down drift areas

of the shoreline?

Should a study of these points indicate the desirability of using a groin system, then it must be given the same engineering consideration in siting and details of construction as any other engineering work.

(1) Length. Model tests, described in Ref. 50, have shown that when waves plunge, sand is carried shoreward and the beach steepens while spilling breakers generally tend to flatten a beach. To be more specific, the maximum drift toward the beach was found to occur for waves having a deep water steepness between 0.01 and 0.02. As the end of a groin approaches the breaking point of waves within this limit, scour increases on the updrift side of the groin and at the end with increased deposition on the down drift side. As the end is moved shoreward, these effects decrease. The model tests showed that the best balance between these effects giving maximum deposition and minimum scour occurred for a groin length equal to 40% of the distance to the plunge point of the critical waves. It is recommended that these experimental results be utilized in determination of groin length based on observations of the plunge point of the predominant waves having a deep water steepness between 0.01 and 0.02. To minimize the undesirable effects of a groin system on the down drift shoreline, a device known as "groin shortening" should be used. This consists simply of shortening the groins at the down drift end of a system. Usually an angle of 6° with the line connecting the groin ends is laid off to intersect the down drift shoreline. The groin length is then cut back to this line and the spacing is reduced to maintain the normal ratio of spacing to length. This widely used procedure will insure that the littoral current runs smoothly toward the unprotected coastline.

As a general rule, for economy and reduction of damage to down drift areas, a groin should only be long enough to create the desired stabilization of the shoreline.

(2) Groin Spacing. The factors of length and spacing interact to determine the shape of the equilibrium arc which will form between adjacent groins. This is an important point since the updrift groin must be protected from lee side scour and eventual flanking at the inshore end. Where the inshore end is anchored to the sea wall, it is obvious that the arc of equilibrium must be maintained with sufficient beach in front of the sea wall to insure against undermining. The alignment of the arc of equilibrium can be determined as outlined in Chapter III based on a refraction diagram for the mean wave condition, i.e., the wave condition which produces the dominant direction and net rate of littoral transport. As a guide to groin spacing, model tests have shown that for waves approaching the shore at an angle of incidence between 60° and 90°, which is the usual case, a spacing of 4 times the length of the groin seaward of the shoreline gave the greatest deposition. For lower angles of incidence, a spacing of 3 times the length proved to be optimum. A rule of thumb for determining spacing given in Ref. 49 is as follows:

(a) Determine alignment of beach profile immediately adjacent to and updrift from existing nearby impounding structures.

(b) Project line parallel to this from the outer end of the anticipated beach fillet at the down drift groin. Intersection of this line with the existing shoreline locates base of the next groin.

Rules such as the above must be used with great caution and only as a preliminary determination of groin spacing. Establishment of the equilib-

rium arc should provide the final answer on this problem. A more detailed discussion of the interrelation between length and spacing is contained in Ref. 4.

(3) Alignment. While various alignments have been tried, the most efficient and most economical groins have proved to be those aligned normal to the shoreline. Curved, hooked and angle groins as well as those having "T", "L", or "Z" alignments are generally subject to much greater scour than the simple, straight structure, and for that reason are not recommended. Where a major realignment of the shore is anticipated as a result of groin construction, it may be desirable to align the structures normal to this future shoreline.

(4) Groin Profile. The inshore section of a groin must be firmly anchored in the sea wall or upland which it is to protect. The height of this section, extending from the sea wall to the crest of the anticipated beach berm, should be such that by-passing of littoral materials will occur during storms and thus replenish the downdrift compartment at its most vulnerable point. Excessive groin height in the inshore section is one of the most frequent causes of failure of a sea wall-groin system. A height in this area equal to the height of high water which occurs frequently plus the height of normal wave run-up is recommended. Beyond the crest of the anticipated berm, the profile varies with the type of structure. Rubble mount "notched" groins are being used extensively on the New Jersey coast. These groins slope downward from +10.0 at the inshore section to +2.0 at the notch located approximately 350' seaward of the beach crest, then rise abruptly to +7.0 and are level throughout their outer length. These pre-

ceding heights are measured from the mean low water datum (MLW). Other groin types frequently exhibit a sloped section from the height at the berm crest parallel to the estimated slope of the beach until mean low water level is reached. Beyond this point the outer section is level to the end. The advantages of this sloped arrangement are obvious from a material economy standpoint; however, the New Jersey stone groin has the advantage of easy access for repairs.

(5) Groin Types. In general, groins are classified as either permeable or impermeable and the discussion of relative merit is far from being complete. Within these general classifications, groins are typed according to construction material as follows: timber, stone, concrete, steel sheet piling, etc. The variation in groin design even within one of these types is astonishing and the reader is referred to the many illustrations contained in current engineering literature.

Selection of type and design depends on the following factors:

(a) Wave exposure and resulting wave forces.

(b) Foundation conditions. An adequate subsurface exploration is essential.

(c) Material availability and resistance to particular local conditions. The earlier brief discussion on materials for sea walls applies also to groins.

(d) Comparison of annual costs for the various possible types. This should include depreciation, interest on investment and annual expenditures for maintenance.

In designing a groin the following general requirements must be fulfilled:

(a) It must resist wave attack and must not contribute to self-destroying erosion.

(b) It must be stable. This requires a calculation of the earth forces which will be created by an effective groin and of the possibility of exceeding the shear strength of foundation materials.

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